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NORTHEAST FLOOD STUDIES

INTERIM REPORT
ON
REVIEW OF SURVEY

ANSONIA - DERBY LOCAL PROTECTION

NAUGATUCK RIVER

CONNECTICUT

U.S. Army Engineer Division, New England
Corps of Engineers Waltham, Mass.

13 APRIL 1960

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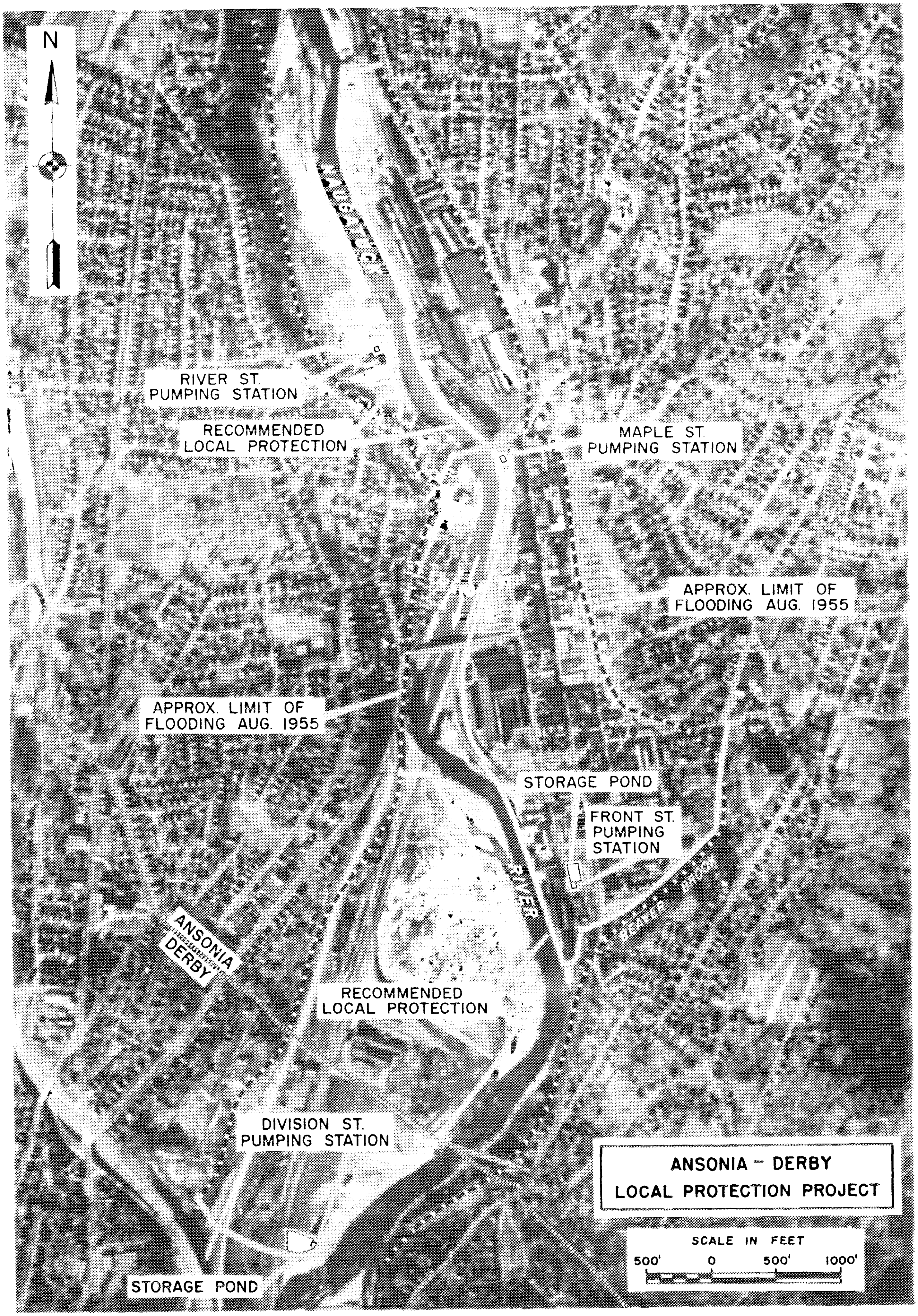
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FLOODING AUG. 1955
STORAGE POND
FRONT ST.
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BEAVER BROOK
ANSONIA
DERBY
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DIVISION ST.
PUMPING STATION
STORAGE POND
ANSONIA – DERBY
LOCAL PROTECTION PROJECT
SCALE IN FEET
500' 0 500' 1000'

SYLLABUS

The Division Engineer finds that there is urgent need for modification of the existing plan for flood control in the Naugatuck River basin in order to insure the stability of present development, the security of the inhabitants, and the preservation of economic values, and to permit utilization of currently vacant land and structures for which a strong demand exists. He finds that the Naugatuck River produces major damages in highly industrialized and commercialized areas of Ansonia and a contiguous area of Derby, Connecticut and that these areas would still suffer severe damages in a recurrence of the maximum flood of record even after construction of the authorized and recommended reservoirs in the Naugatuck River basin, a sub-basin of the Housatonic River Basin.

The Division Engineer recommends construction of a local protection project in Ansonia and Derby at an estimated total first cost of $6,020,000. He further recommends that local interests be required to (1) provide, without cost to the United States, all lands, easements, and rights-of-way necessary for the construction and operation of the project, including lands for spoil disposal areas, pumping stations, and drainage systems; (2) hold and save the United

States free from damages due to the construction works; (3) maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of the Army; (4) permit no encroachment on the improved channels or on the ponding areas and, if ponding areas and/or capacities are impaired, to provide substitute storage capacity or equivalent pumping capacity promptly without cost to the United States; and (5) contribute to the United States 14.9 percent of the construction cost, which contribution is presently estimated at $850,000, said amount being based on enhancement benefits accruing to the project and the additional cost of the plan preferred by local interests over the plan found by the Division Engineer to be the most feasible. The Federal first cost of the project is currently estimated at $4,870,000; the first cost to local interests for lands and relocations, in addition to the cash contribution, is currently estimated at $300,000.

INTERIM REPORT ON REVIEW OF SURVEY

ANSONIA-DERBY LOCAL PROTECTION
NAUGATUCK RIVER, CONNECTICUT

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U. S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM 54, MASS.

ADDRESS REPLY TO:
DIVISION ENGINEER

REFER TO FILE NO.

NEDGW

13 April 1960

SUBJECT: Interim Report on Review of Survey for Flood Control, Ansonia-Derby Local Protection, Naugatuck River, Connecticut

TO: Chief of Engineers
Department of the Army
Washington, D. C.
ATTENTION: ENGCW-P

SECTION I - AUTHORITY

1. AUTHORIZING RESOLUTIONS

This report is submitted pursuant to authority contained in the following Congressional resolutions which are quoted in part:

a. Resolution by the Committee on Public Works of the United States Senate, adopted September 14, 1955:

> That the Board of Engineers for Rivers and Harbors, created under Section 3 of the River and Harbor Act, approved June 13, 1902, be, and is hereby, requested to review previous reports on the . . . Housatonic River, Connecticut, Massachusetts and New York . . . in the area affected by the hurricane flood of August 1955, to determine the need for modification of the recommendations in such previous reports and the advisability of adopting further improvements for flood control and allied purposes in view of the heavy damages and loss of life caused by such floods.

b. Resolution by the Committee on Public Works of the House of Representatives, adopted June 13, 1956:

That the Board of Engineers for Rivers and Harbors be, and is hereby, requested to review the reports on the Housatonic River, Connecticut, Massachusetts, and New York, published as House Document 338, 77th Congress, and other pertinent reports, with a view to determining what improvements for flood control are advisable at this time, with particular reference to the following areas and locations: Naugatuck River Basin for protection at Derby-Ansonia, ... Connecticut, ...

<u>c</u>. <u>Resolution by the Committee on Public Works of the House of Representatives,</u> adopted July 23, 1956:

That the Board of Engineers for Rivers and Harbors be, and is hereby, requested to review the reports on the Housatonic River, Connecticut, Massachusetts, and New York, published as House Document 338, 77th Congress, and other reports, with a view to determining the advisability of providing improvements in the interest of flood control and allied purposes on the Naugatuck River, with particular reference to . . . . Ansonia, and Derby, Connecticut, at this time.

2. ASSIGNMENT OF STUDY

<u>a</u>. In letter dated September 14, 1955, the Chairman of the Committee on Public Works of the United States Senate transmitted the foregoing Senate Resolution to the Chief of Engineers and requested appropriate attention. By first indorsement dated September 16, 1955, the Chief of Engineers assigned the study and the preparation of a report to the Division Engineer, U. S. Army Engineer Division, New England.

<u>b</u>. In letters dated June 18, 1956 and July 25, 1956, respectively, the Chairman of the Committee on Public Works of the House of Representatives transmitted the foregoing House Resolutions to the Chief of Engineers. By first indorsements dated, respectively, July 26, 1956 and August 3, 1956, the Chief of Engineers referred the resolutions to the Division Engineer, U. S. Army Engineer Division, New England.

SECTION II - SCOPE

3. SCOPE OF REPORT

This interim report of survey scope comprises a review of the flood problems caused by the Naugatuck River in Ansonia and a small, contiguous area in Derby, Conn. It is one of a series of reports which, when completed, will constitute a review of the needs of the New England area with respect to flood control and allied water uses. Flood problems and solutions considered for the remainder of the Naugatuck River Basin are included in previous reports cited in paragraph 5, following. The purpose of this report is to determine the advisability and economic feasibility of flood control improvements in the area under consideration and to make specific recommendations.

4. SCOPE OF STUDIES

a. Topographic surveys. U. S. Army Map Service, U. S. Geological Survey, State and local maps, and plane table topographic surveys of the project area were used in the study.

b. Site explorations. Subsurface investigations consisted of field reconnaissance by geologists and soils engineers, excavation of foundation test pits, and review of previous borings for bridge foundations in the area.

c. Economic investigations. Surveys of experienced flood damages, made after the floods of 1938, 1948, and 1955, were reviewed in 1958. An investigation of potential higher utilization in the project areas was also made in 1958. The surveys consisted of field examination of the project areas and personal interviews with municipal and State officials, officials of industrial concerns, and individuals experiencing losses.

d. Office studies. Office studies consisted of hydrologic and hydraulic analyses and determination of approximate quantities and costs of the major items of construction, utility relocations and real estate required for the project.

e. Consultations with interested parties. A public hearing was held in Waterbury, Conn., on December 11, 1956, at which time interested parties requested consideration of improvements in various areas in the Naugatuck River basin, including the Ansonia-Derby area.

A synopsis of the hearing is given in Section XIII. Meetings have been held with State and local officials, the Naugatuck Valley River Control Commission, and with individuals.

f. Field reconnaissance. Field reconnaissance of the problem area and sites of potential improvement has been made by the Division Engineer and his staff.

SECTION III - PRIOR REPORTS

5. PUBLISHED REPORTS

Flood control in the Naugatuck River Basin has been considered in the following published reports on the Housatonic River:

a. "308" Report. A report dated June 25, 1931 and printed as House Document No. 246, 72d Congress, 1st session, covered navigation, water power, flood control, and irrigation in the Housatonic River basin. The report found that further improvements were not warranted at that time.

b. 1940 Report. A report dated June 20, 1940 and printed as House Document No. 338, 77th Congress, 1st session, recommended construction of the Thomaston Dam on the Naugatuck River above Thomaston, Conn. This project was authorized by Public Law 534, 78th Congress, 2d session, approved December 22, 1944.

c. NENYIAC Report. Flood control and allied water uses are considered in Part 2, Chapter XXII, "Housatonic River Basin," of The Resources of the New England-New York Region. This comprehensive report presented an inventory of the resources of the New England-New York area and recommended a master plan to be used as a guide for the regional planning, development, conservation, and use of land, water, and related resources of the region. Proposals to reduce flood losses were also included. Prepared by the New England-New York Inter-Agency Committee, the report was submitted to the President of the United States by the Secretary of the Army on April 27, 1956. Part 1 and Chapter 1 of Part 2 are printed as Senate Document 14, 85th Congress, 1st session.

d. 1956 Interim Report. An interim report dated May 31, 1956 and printed as House Document No. 31, 85th Congress, 1st session, reviewed the need for additional flood control works on the upper Naugatuck River upstream from the authorized Thomaston Reservoir. The report recommended that the authorized plan for flood control in the Housatonic River be modified to provide for the construction of two flood control dams and reservoirs upstream of Torrington, Conn., one on Hall Meadow Brook, a tributary of the West Branch of the Naugatuck River, and one on the East Branch of the Naugatuck River. These projects were authorized by the Flood Control Act of 1958 (Public Law 85-500, 85th Congress), approved July 3, 1958.

e. 1958 Interim Report. An Interim Report, dated June 30, 1958, reviewed the need for additional flood control works in the Naugatuck River downstream from the authorized Thomaston Reservoir. The report recommended that the authorized plan for flood control in the Housatonic River be modified to provide for the construction of four flood control dams and reservoirs on tributaries of the Naugatuck River downstream from the authorized Thomaston Reservoir. This report has been concurred in by the Board of Engineers for Rivers and Harbors and the Chief of Engineers and has been transmitted to the Bureau of the Budget for their comments prior to submission to Congress.

SECTION IV - DESCRIPTION

6. LOCATION AND EXTENT

Ansonia is located on the Naugatuck River approximately two miles upstream from the confluence of the Naugatuck and Housatonic Rivers in New Haven County, Conn. Located in southern Connecticut, Ansonia is 10 air miles west of New Haven, 35 miles southwest of Hartford, and 10 miles north of Long Island Sound. The city, which is coextensive with the town of Ansonia, covers an area of 6.3 square miles, of which 5.0 square miles is on the east bank and 1.3 square miles on the west bank of the Naugatuck River. The built-up area of the city covers approximately 1.3 square miles, divided almost equally on each bank of the river. A contiguous area of the city of Derby, also considered in this report, consists of 42 acres of industrial and commercial land located downstream of Ansonia on the west bank of the river. Plate No. 1 shows the relative locations of Ansonia and Derby in the Naugatuck River basin. A general map of Ansonia and the adjacent Derby area is shown on Plate No. 2.

7. TOPOGRAPHY

The area is hilly, with wooded hilltops and cleared valleys and flood plains, the latter including the major portion of the built-up sections of the two cities. Elevations range from less than 20 feet near the river to over 500 feet above mean sea level in the northeasterly corner of Ansonia. Topography of the Ansonia-Derby area is shown on U. S. Army Map Service map, "Ansonia, Conn.," to a scale of 1:25,000 with 20-foot contour interval and on U. S. Geological Survey map "Ansonia, Conn.," to a scale of 1:31,680 with 10-foot contours.

8. GEOLOGY

a. Structural. Ansonia lies on the western highland bordering the Connecticut structural valley. The bedrock of the highland consists of a northeast-striking series of folded schists, classified locally as Lower Paleozoic periphyritic gneiss and consists of bands of gneiss intercalated between the schists.

b. Physiography. Ansonia is situated in the Naugatuck River valley, two miles above the river's confluence with the Housatonic River, and lies at the inshore limit of oceanic tidal influence. The city occupies the valley bottom at this point and partly ascends the adjacent hills which attain heights of 400 to 500 feet. The Naugatuck River at Seymour, four miles north, has a narrow valley which was cut into the western highland since the regional uplift in Tertiary time. Southward, the valley widens until, at Ansonia, sufficient breadth is attained to have permitted the formation of temporary glacial lake sand and gravel deposits over a mile in breadth. Within this lake bed, extending southward from over a mile north of the business center to the Housatonic River, is a deposit of river alluvium, largely gravel, which is about one-half mile in breadth, bordering and underlying the river. The hills surrounding these glacial lakes and recent alluvial deposits are capped with ground moraine consisting of sandy till.

9. STREAM CHARACTERISTICS

a. Main stream. The Naugatuck River is formed in the city of Torrington, Conn., by the confluence of its West and East Branches at an elevation of approximately 525 feet above mean sea level. The drainage area at the Maple Street bridge in Ansonia is approximately 306 square miles. The river falls about 18 feet in the 2-1/2-mile reach through Ansonia, with the lower end being affected by tidal fluctuations.

b. Tributaries. The only tributary stream of any significance in the Ansonia area is Beaver Brook. This stream has a drainage area of about 3.6 square miles and is 3 miles long, rising in Peat Swamp Reservoir in Seymour, to the north of Ansonia. The stream flows through the easterly portion of Ansonia and through the southerly portion of the built-up area of the city, emptying into the Naugatuck River near the southerly limit of the city. The stream has a total fall of about 300 feet.

SECTION V - ECONOMIC DEVELOPMENT

10. POPULATION

Over three-quarters of the area and almost the entire population of Ansonia and about one-third of the area and two-thirds of the population of Derby are within the Naugatuck River basin. Based on U. S. Census figures for 1950, that part of the population of the two cities which is included in the basin totals 25,172, with 18,332 in Ansonia and 6,840 in Derby.

11. TRANSPORTATION

a. Commercial facilities. The cities of Ansonia and Derby are served by the New Haven Railroad, major bus lines, and numerous trucking companies. There is a small, privately owned and operated airfield in the area. Commercial airlines stop at New Haven approximately 10 miles east of Ansonia, and at Bridgeport, approximately 12 miles south of Ansonia.

b. Highways. Ansonia and Derby are linked to a network of paved roads crisscrossing the Naugatuck River basin. Most important of these is State Route 8 which passes through both cities in its course along the entire length of the Naugatuck River.

12. MANUFACTURING

a. Extent of manufacturing. The cities of Ansonia and Derby constitute a heavily industrialized area typical of the urban areas of the Naugatuck Valley. Approximately 1 of every 4 persons living in the area is employed in manufacturing.

b. Leading industries. The most important industrial activity in the area is the production of fabricated metals, with about 70 percent of all manufacturing workers engaged in this industry. Next in importance is the machinery and machine tool industry, which employs over 20 percent. The largest plant of one of the nation's leading producers of fabricated brass, copper, and bronze, is located in Ansonia.

c. Important products. The Ansonia-Derby area produces a proportionately large percentage of the non-ferrous metal products of the Naugatuck River basin, which is the source of one-third of the United States' brass and bronze products. Approximately 2,800 workers in Ansonia are employed in the manufacture of brass, copper, and bronze in sheet, wire, rods, or tubes; over 1,000 in the production of rolls, castings, weldments, heavy machinery, gears, gear drives, gear machines, machine tools, and flexible couplings; nearly 500 in the manufacture of screw machine products, metal stampings, and light assemblies; and about 300 in the processing of electrical products. In Derby, over 900 workers are employed in the fabrication of brass, copper, and bronze; about 350 in the gas and electric industry; and over 300 in printing and publishing.

SECTION VI - CLIMATOLOGY

13. TEMPERATURE

The climate of the Naugatuck River basin is generally moderate. In the southern portion, extremes of temperature are tempered by the influence of nearby Long Island Sound. The average annual temperature is about 47°F, ranging from 50°F in the southern part to about 45°F in the headwaters. Freezing temperatures can be expected from the middle of November until the end of March.

14. PRECIPITATION

The basin is subject to frequent but short periods of heavy precipitation which are produced by convective and cyclonic disturbances, and occasionally by coastal storms, some of which are of tropical origin. The average annual precipitation over the Naugatuck

River basin is about 50 inches, uniformly distributed throughout the year. The annual snowfall over the watershed varies from about 35 inches near the coast to over 80 inches in the headwater region. The water content of the snow cover in the early spring often totals 4 to 6 inches.

SECTION VII - RUNOFF AND STREAMFLOW DATA

15. GAGING STATION RECORDS

The U. S. Geological Survey has published records of river stages and streamflows at four locations in the basin for various lengths of time since 1918. The records are generally good to excellent, except during periods of ice when they are fair. The locations of stream-gaging stations in the basin, together with their respective drainage areas and periods of record, are listed in Table 1.

TABLE 1

GAGING STATIONS - NAUGATUCK RIVER BASIN

Location	Drainage Area (sq. miles)	Period of Record
Naugatuck River near Thomaston, Conn.	71.9	1930 - 1960
Leadmine Brook near Thomaston, Conn.	24.0	1930 - 1960
Naugatuck River near Naugatuck, Conn.	246	1918 - 1924 1929 - 1955
Naugatuck River near Beacon Falls, Conn.	261	1955 - 1960

SECTION VIII - FLOODS OF RECORD

16. HISTORIC FLOODS

The earliest recorded significant flood in the Naugatuck River basin occurred in February 1691. Two large floods were recorded in November 1853 and April 1854. The flood of October 1869 was the greatest prior to 1900, with other serious floods occurring in 1874, 1888, 1891, 1896, and 1897. There is no reliable information on the magnitudes of any of these floods.

17. RECENT FLOODS

The Naugatuck River basin has experienced 6 major floods within the past 30 years. These floods are briefly described in the following paragraphs, with a summary of comparative flood magnitudes of the three largest indicated in Table 2 on page 12.

a. November 1927. This flood resulted from 5.5 inches of rainfall during November 3 and 4 on ground saturated by rains during the previous month.

b. March 1936. This flood was caused by four distinct storm centers that passed over the northeastern states between March 9 and 22. The runoff from the rains was augmented by considerable snowmelt.

c. September 1938. This flood resulted from the heavy rainfall that accompanied a tropical hurricane which passed over New England on September 21. The rain fell on ground saturated by rains earlier in the month. The average rainfall over the Naugatuck River basin during this storm exceeded 10 inches.

d. December 1948. This flood resulted from about 9 inches of rain falling on frozen ground. The runoff was augmented by snowmelt.

e. August 1955. The greatest flood of record was caused by rainfall that preceded and accompanied hurricane Diane. Rainfall, which averaged more than 13 inches in the upper watershed and 10 inches in the lower part of the basin, followed more than 7 inches of rain from hurricane Connie the previous week.

f. October 1955. This flood resulted from a storm that moved up the Atlantic Coast from Florida and deposited 10 to 14 inches of rainfall over the lower half of the Naugatuck River basin.

18. FLOOD CHARACTERISTICS

The more critical floods, which, as noted, can occur in any month of the year, develop from rainfall alone where the intensity of the rainfall, rather than the total volume, may determine the magnitude of the flood peaks. The quick development of floods is due to the short, steep tributaries which empty into the main channel almost concurrently. This is illustrated by the fact that, in major floods, the Naugatuck River has crested along its entire length within a period of 5 to 8 hours.

In the lower reaches of the Naugatuck River, water surface elevations are affected by tidal conditions in Long Island Sound as well as by concurrent flow in the Housatonic River. Critical flood conditions in the Ansonia area can result from flood flows in the Naugatuck River and also from the synchronization of Naugatuck River flow with high stages in the Housatonic River caused by either ocean tide or high flow in the river.

SECTION IX - STANDARD PROJECT FLOOD

19. GENERAL

The standard project flood is a synthetic flood used to measure the flood potentialities of a river basin and as criteria for establishing design grades for walls and dikes in local protection projects. It represents flood discharges that may be expected from the most severe combination of meteorological and hydrologic conditions that are considered reasonably characteristic of the geographical region, excluding extremely rare combinations.

20. NAUGATUCK RIVER

The magnitude of the August 1955 flood made it necessary to reappraise the flood potential of the Naugatuck River basin. Analyses of the 1955 flood indicated a substantial increase in the magnitude of unit hydrographs previously used for determination of the standard project flood in this basin, particularly for intense storms following an antecedent wet period. Magnitudes of the standard project flood (SPF) and major floods of record at Ansonia, Connecticut, and reductions effected by authorized and recommended reservoirs are shown in Table 2.

TABLE 2

EFFECT OF PROPOSED RESERVOIR SYSTEM IN REDUCING RECENT MAJOR FLOODS AND STANDARD PROJECT FLOOD - NAUGATUCK RIVER AT BRIDGE STREET, ANSONIA, CONN.
(Drainage area = 311 sq. mi.)

Flood	Natural		Modified*	
	Stage in ft.	Discharge in c.f.s.	Stage in ft.	Discharge in c.f.s.
December 1948	16.6	32,700	12.2	15,600
August 1955	22.0	112,000	18.1	54,000
October 1955	17.3	40,000	14.1	23,000 (est.)
SPF	23.6	148,000	19.7	75,000

*The proposed system includes the following reservoirs:

Thomaston
Hall Meadow Brook
East Branch
Northfield Brook
Black Rock
Hancock Brook
Hop Brook

21. BEAVER BROOK

The standard project flood developed for Beaver Brook was based on a standard project storm and a synthetic unit hydrograph derived from constants obtained from comparable nearby watersheds. The rainfall pattern was arranged to give the most critical runoff conditions and then applied to the 3-hour unit hydrograph, resulting in a flood hydrograph with a peak ordinate of 3,180 c.f.s.

SECTION X - EXTENT AND CHARACTER OF THE FLOODED AREA

22. GENERAL

Highly developed industrial, residential, and commercial areas along both banks of the Naugatuck River in Ansonia and the adjacent Division Street section of Derby are affected by flooding. Nearly 400 acres in Ansonia and 40 acres in the portion of Derby under consideration are vulnerable to flooding by the standard project flood modified by authorized and recommended reservoirs. These flood-prone areas constitute an important portion of the major nonferrous metal manufacturing region in the nation. Manufacture of heavy machinery, castings, wire, and cables; fabrication of screw machine and copper base products; and printing are the principal industrial activities in this populous area. The area flooded in the record flood of August 1955 is shown on Plate No. 2.

Largest of the lower Naugatuck flood problem areas is the Ansonia east bank section bounded by Liberty Street, East Main Street, and Beaver Brook. Thirteen industrial firms, including the extensive American Brass and Farrel-Birmingham companies, and the heart of Ansonia's business district are located in this area. With payrolls of over $15 million annually, these plants employ about 70 percent of the city's industrial labor force and manufacture metal, lumber, textile and printed products worth about $50 million annually. Two-thirds of the 150 dwellings subject to flooding in Ansonia are located in the southern portion of this area. Urban renewal of a portion of this east bank area is currently being studied by the Ansonia Redevelopment Agency. Implementation of this program is dependent upon favorable consideration of flood protection for the area. Relatively few changes are planned in the east bank business district upstream of Bridge Street or in the riverside industrial area.

A second section severely affected by flooding is the Ansonia-Derby area on the west bank in the vicinity of Division Street. Major firms in this lowland area are the Hershey Metal Products Corporation, manufacturer of screw machine products, and the Charlton Press Corporation, which publishes magazines. Industrial payrolls in this area exceed $2-1/2 million annually and the value of products made at these plants is estimated at $7 million annually.

Other properties in this area affected by flooding are a drive-in theater, a commercial block, and the Ansonia freight yards of the New Haven Railroad.

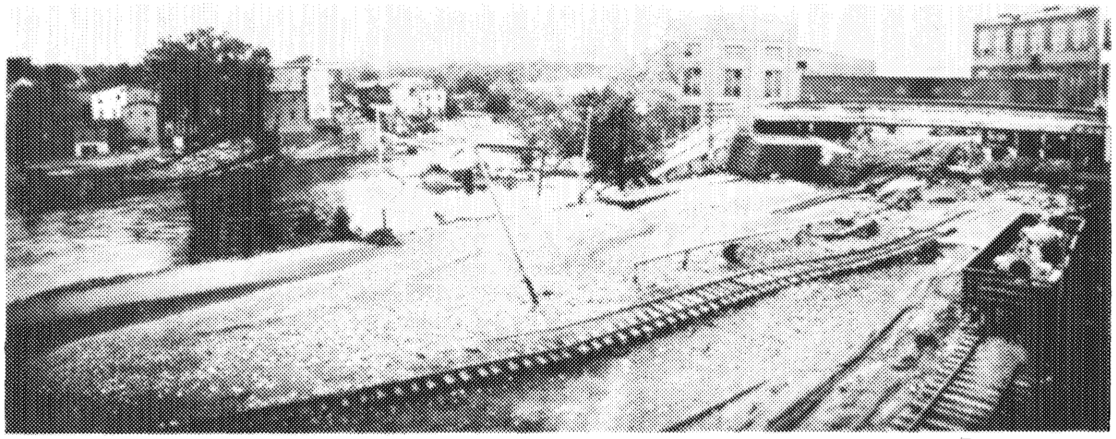
A third section vulnerable to flooding is the Maple and Broad Streets area along the west bank in the northern part of Ansonia. The principal concentration of loss lies in the industrial district north of Maple Street where industrial payrolls total about $1 million. Nearly $3 million worth of brass screw machine products, rayon velvets, and electrical cables are produced annually or stored in this flood-prone area. Largest factories in the area are the Ansonia Manufacturing Company and the Velvet Corporation of America. Under provisions of the Federal Urban Renewal Act, local officials plan, as part of a redevelopment program in the Broad Street area, to bring a low portion of the commercial and warehouse area south of Maple Street to an elevation just above the 1955 stage modified by authorized and recommended reservoirs. A public housing development, in keeping with the residential character west of Broad Street, is proposed for the Broad Street area.

SECTION XI - FLOOD DAMAGES

23. FLOOD OF AUGUST 1955

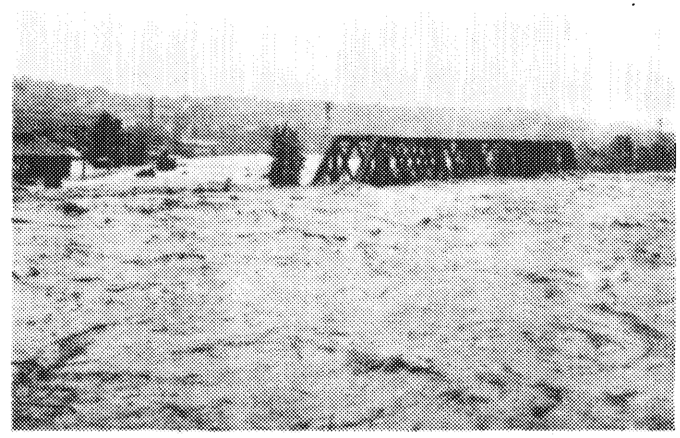
The flood of August 1955 on the lower Naugatuck River claimed two lives in the Ansonia area. Cresting some 9 feet above the New Year's flood of 1948-49, the former record flood, the August 1955 flood caused an estimated loss of nearly $34 million in Ansonia and the Division Street section of Derby. Damage to fourteen industrial plants accounted for nearly three-fourths of the total loss. Most of the remaining loss occurred in the central business district of Ansonia on the east bank near Main and Bridge Streets. Typical flood scenes are pictured on page 15.

A detailed investigation conducted shortly after the August flood indicated that over 440 buildings were damaged in the Ansonia area. Nearly 150 dwellings, 270 commercial establishments, and 6 public buildings were flooded to depths up to 13 feet. Serious urban losses were experienced on the east bank in the central business district near Bridge Street and in the residential area between Central Street and Beaver Brook. Over 70 buildings were condemned because of



Treskunoff

View upstream toward site of demolished Maple Street bridge



Helen Martin

View downstream toward railroad bridge

Charles Hartman

Intersection of Main and Bridge Streets
before flood reached its peak

Franklin Farrell III

Water gushing from manholes and
cellars on Main Street

FLOOD SCENES IN ANSONIA, CONN., AUGUST 1955 flood damage, mainly in the west bank Broad Street area and the east bank Beaver Brook area.

Industrial losses in the area totaled over $25 million. Four of the seven firms with flooding above the basement level suffered crippling losses when up to 12 feet of water roared through their plants. The two largest firms, the Ansonia divisions of the American Brass and Farrel-Birmingham companies, experienced major structural, machinery and stock losses in their east bank plants near Maple and Bridge Street bridges. Major industrial losses were also suffered by the Hershey Metal Products and Charlton Press corporations which are located on the west bank near the Division Street bridge. Floating debris and silt deposits contributed to extensive destruction of machinery, electric motors and finished products throughout the flood area. Lengthy and costly overhaul and cleanup measures were required in many plants before normal operations could be resumed.

Destruction of the Maple and Division Street bridges and the American Brass Company private bridge and damage to the Bridge Street bridge caused major transportation delays and hindered repair of severed utility lines. Nearly five miles of track and the abutments of three bridges on the Naugatuck branch of the New Haven Railroad were seriously damaged. Table 3 shows the 1955 experienced flood losses in the area, tabulated by zones and types of loss.

TABLE 3

EXPERIENCED AUGUST 1955 FLOOD LOSSES
LOWER NAUGATUCK RIVER
ANSONIA - DERBY AREA

Type Loss	East Bank	West Bank	Total
Urban	$ 5,640,000	$ 520,000	$ 6,160,000
Industrial	13,390,000	11,630,000	25,020,000
Utility	120,000	-	120,000
Highway	1,590,000	210,000	1,800,000
Railroad	380,000	380,000	760,000
Totals	$21,120,000	$12,740,000	$33,860,000

24. FLOOD OF OCTOBER 1955

The October 1955 flood was the second greatest flood of record in the Ansonia area. Because this flood followed close on the heels of the August flood, while restoration was still under way and an atmosphere of flood-consciousness prevailed, it caused less havoc than it would have otherwise. No survey of October damages was made, but it is estimated that losses amounted to over $2 million.

25. RECURRING LOSSES

Under economic conditions existing in 1959, a recurrence of the August 1955 flood, after stage reductions effected by the authorized and recommended reservoir system, would cause an estimated loss of over $13.7 million. Approximately $9.4 million of this loss would occur along the east bank and $4.3 million along the west bank. Residual losses after reductions by the reservoir system would be negligible in two areas - a small area on the east bank immediately downstream of Beaver Brook and the Broad Street area on the west bank where urban renewal is under way.

A summary of recurring August 1955 losses by types of damage is presented in Table 4.

TABLE 4

RECURRING AUGUST 1955 FLOOD LOSSES
AFTER REDUCTIONS BY AUTHORIZED
AND RECOMMENDED RESERVOIRS

LOWER NAUGATUCK RIVER
ANSONIA-DERBY AREA
(Jan. 1960 Price Level)

Type Loss	East Bank	West Bank	Total
Urban	$1,510,000	$ 50,000	$ 1,560,000
Industrial	7,640,000	4,240,000	11,880,000
Utility	20,000	-	20,000
Highway	160,000	10,000	170,000
Railroad	80,000	10,000	90,000
Totals	$9,410,000	$4,310,000	$13,720,000

26. AVERAGE ANNUAL LOSSES

Estimated recurring losses in the Ansonia area were converted to average annual losses by correlation of stage-damage, stage-discharge and discharge-frequency relationships to produce damage-frequency relationships in accordance with standard practice of the Corps of Engineers. Average annual losses remaining in the area after stage reductions effected by the reservoir system amount to $288,000.

SECTION XII - EXISTING FLOOD CONTROL PROJECTS

27. CORPS OF ENGINEERS' PROJECTS

There are no existing Corps of Engineers' flood control projects in Ansonia. Authorized and recommended flood control reservoirs in the Naugatuck River basin, which would effect flood reductions in Ansonia, are discussed below.

28. AUTHORIZED RESERVOIRS

a. Thomaston Dam and Reservoir. The Thomaston Dam, authorized by the Flood Control Act approved December 22, 1944 (Public Law 534, 78th Congress, 2d session), is located on the main stem of the Naugatuck River about 28 miles upstream from Ansonia. The project provides for construction of a rolled earth and rockfill dam 2,000 feet long, rising 142 feet above the stream bed. At spillway crest, the reservoir will extend upstream approximately 6-1/2 miles and will have a storage capacity of 42,000 acre-feet, all reserved for flood control, equivalent to 8.1 inches of runoff from the tributary drainage area of 97 square miles. A contract for railroad relocation made necessary by the project was awarded in October 1957. A contract for construction of the dam and appurtenant works was awarded in April 1958, with the project scheduled for completion in December 1960. Estimated costs, as of the last printed Annual Report of the Chief of Engineers (1958) are $4,621,000 for construction and $11,279,000 for lands and damages, including highway, railroad, and utility relocations, a total of $15.9 million for new work.

b. Hall Meadow Brook Dam and Reservoir. The Hall Meadow Brook Dam, authorized by the Flood Control Act of 1958 (Public Law 85-500, 85th Congress), will be located on Hall Meadow Brook, a tributary of the West Branch of the Naugatuck River, about 5 miles upstream from Torrington, Connecticut and about 15 miles upstream from the Thomaston Dam. At spillway crest, the reservoir will have a storage capacity of 7,200 acre-feet, equivalent to 11.1 inches of runoff from the tributary drainage area of 12.2 square miles. The estimated first cost (1956 price level) is $2,420,000, of which the Federal first cost is $1,960,000. The project is currently under design. Consideration is being given in the design stage to diversion of Reuben Hart Brook, a small tributary which presently empties into Hall Meadow Brook just downstream of the authorized project. The drainage area controlled would thereby be increased to 17.2 square miles.

c. East Branch Dam and Reservoir. The East Branch Dam, also authorized by the Flood Control Act of 1958, will be located on the East Branch of the Naugatuck River, about 2 miles upstream from Torrington and about 12 miles upstream from the Thomaston Dam. At spillway crest, the reservoir will have a storage capacity of 5,150 acre-feet, equivalent to 10.5 inches of runoff from the tributary drainage area of 9.25 square miles. The estimated first cost (1956 price level) is $2,670,000, of which the Federal first cost is $1,780,000. No work has been initiated on this project.

29. RECOMMENDED RESERVOIRS

In report dated 30 June 1958, the Division Engineer recommended construction of four additional flood control reservoirs on tributaries of the Naugatuck River downstream from the Thomaston Dam and upstream from Ansonia. This report has been concurred in by the Board of Engineers for Rivers and Harbors and the Chief of Engineers and has been transmitted to the Bureau of the Budget for their comments prior to submission to Congress. Projects recommended in the report are described below.

a. Northfield Brook Dam and Reservoir. This project provides for construction of an earth fill dam on Northfield Brook about 28 miles upstream from Ansonia. The reservoir, at spillway crest, would have a storage capacity of 2,430 acre-feet, equivalent to 8 inches of runoff from the tributary drainage area of 5.7 square miles. The estimated first cost (1958 price level) is $1,620,000.

b. Black Rock Dam and Reservoir. The project provides for construction of an earth fill dam on Branch Brook about 26 miles upstream from Ansonia. The reservoir, at spillway crest, would have a storage capacity of 8,860 acre-feet, equivalent to 8 inches of runoff from the tributary drainage area of 20.8 square miles. The estimated first cost (1958 price level) is $3,550,000.

c. Hancock Brook Dam and Reservoir. This project provides for construction of a rockfill dam on Hancock Brook about 22 miles upstream from Ansonia. The reservoir, at spillway crest, would have a capacity of 3,820 acre-feet of storage, equivalent to 6 inches of runoff from the tributary drainage area of 12.0 square miles. The estimated first cost (1958 price level) is $2,520,000.

d. Hop Brook Dam and Reservoir. This project provides for the construction of an earth fill dam on Hop Brook about 13 miles upstream from Ansonia. The reservoir, at spillway crest, would have a storage capacity of 6,840 acre-feet, equivalent to 8 inches of runoff from the tributary drainage area of 16 square miles. The estimated first cost (1958 price level) is $2,600,000.

30. IMPROVEMENTS BY OTHER FEDERAL AND NON-FEDERAL AGENCIES

a. Federal. No projects for flood control in Ansonia have been constructed by other Federal agencies.

b. State of Connecticut. In rebuilding the Maple, Bridge, and Division Streets bridges, the Connecticut State Highway Department, at the suggestion of the Connecticut Water Resources Commission, constructed these bridges with a waterway area capable of passing a flood at least seven times as large as the mean annual flood modified by the Thomaston Reservoir. (In Ansonia, the seven times mean annual flood, modified by Thomaston Dam, is approximately 55 percent of the standard project flood modified by authorized and recommended dams.)

During the 1955 session of its General Assembly, the Connecticut State Legislature passed a river encroachment law. Section 9(c) of Connecticut Public Law No. 364 (1957) directs the State Water Resources Commission to establish lines beyond which, in the direction of the waterways, no obstruction or encroachment shall be placed without approval of the Commission. On the Naugatuck River, these lines generally define the limits of a flood seven times as large as the mean annual flood modified by the Thomaston Reservoir. Such encroachment lines have been established along the Naugatuck River in Ansonia.

SECTION XIII - IMPROVEMENTS DESIRED

31. PUBLIC HEARING

A public hearing with respect to flood control and allied measures for the Naugatuck River and its tributaries was held in Waterbury, Connecticut on December 11, 1956. Approximately 175 persons attended, including representatives of Federal, State and municipal governments, industrial and agricultural interests, civic organizations, and interested individuals. Municipal officials and industrial leaders of Ansonia and Derby expressed the need for additional flood protective measures but did not stipulate specific measures desired. A digest of the public hearing is included as Appendix A of the report on the Naugatuck River dated 30 June 1958 (see preceding par. 5e.).

SECTION XIV - FLOOD PROBLEMS AND SOLUTIONS CONSIDERED

32. FLOOD PROBLEM

In the Ansonia-Derby area, the Naugatuck River is susceptible to floods caused by heavy rains or a combination of heavy rains and melting snow. Runoff from tributaries is rapid due to generally hilly topography. Flooding is aggravated by inadequate river channel and constricted bridge openings. The flood plain on the east bank is almost fully utilized by industrial, commercial, and residential developments. Industrial and commercial properties and the site of the proposed municipal sewage treatment plant are located on the west bank. While the authorized and recommended reservoirs will provide substantial protection to these properties, as shown on Table 2 (page 12), a recurrence of the August 1955 flood, with the reservoirs in operation, would leave a residual estimated loss of nearly $14 million in the Ansonia-Derby area.

33. SOLUTIONS CONSIDERED

Reservoirs, in addition to those authorized and recommended, to further reduce flood stages in Ansonia were considered in the previously submitted report on the Naugatuck River, dated 30 June 1958, and found to lack economic justification.

Channel widening and deepening alone was considered and found to be economically infeasible due to two conditions:

(1) Two of the three highway bridges over the Naugatuck River are new, the previous bridges having been washed out or badly damaged during the 1955 floods. The third bridge is currently being reconstructed. All three bridges are designed to pass a flow equivalent to seven times the mean annual flood modified by the Thomaston Reservoir, which is approximately 55 percent of the standard project flood modified by authorized and recommended reservoirs. The existing railroad bridge has even less waterway area, being able to pass freely only 24 percent of the standard project flood. Raising or adding spans to these four bridges would be inordinately expensive and/or impracticable due to existing developments.

(2) River stages in the lower sections of the Ansonia area are dependent not only upon river discharge but also upon coincident tidal conditions. Backwater effects from tidal action are experienced as far upstream as the railroad bridge under extreme conditions. Channel widening and deepening could not, therefore, wholly solve the flood problem in Ansonia.

Dikes and flood walls, in combination with channel improvement, appear to be the only feasible solution to the flood problem in the Ansonia-Derby area. Along the east bank, existing developments dictate the alignment, with walls being required where existing structures are near the river, and less expensive earth dikes where space permits. The protection studied extends from the mouth of Beaver Brook upstream along the main stem to about 3, 200 feet above Maple Street. Protection along some 2, 600 feet of Beaver Brook would also be required, for which two alternative plans were studied. On the west bank in the vicinity of Division Street, four dike alignments downstream of Division Street and two dike alignments upstream of Division Street were investigated. Protection for an area on the west bank immediately upstream from Maple Street was also studied. The location of the most feasible plan is shown on Plate 2. Plans and details of other plans studied are given in Appendix C.

34. RELATED WATER RESOURCE DEVELOPMENTS

No related water resource development would be feasible in conjunction with the studied plans.

SECTION XV - FLOOD CONTROL PLANS

35. GENERAL

Studies made for this report disclose that flood protection for the Ansonia-Derby area, in addition to that provided by authorized and recommended projects, is needed and is feasible. In these studies, protection by alternative methods, and combinations thereof, was considered. The most feasible plan would protect a total of 114 acres of residential, industrial, and commercial land in two separate areas along the west bank and 118 acres of industrial, commercial, and residential land along the east bank of the Naugatuck River and upstream along Beaver Brook. The location of this plan is shown on Plate No. 2.

36. MOST FEASIBLE PLAN

a. West bank. The plan for protection of the west bank at Division Street would provide for approximately 6,200 feet of dike beginning at the embankment of Route 8 southwest of the Charlton Printing Company in Derby, extending south of a drive-in theater to the bank of the Naugatuck River and upstream along the river channel which would be widened and realigned, to the vicinity of the New Haven Railroad bridge in Ansonia, and tying into high ground to the west. Five stoplog openings, for crossings of the New Haven Railroad tracks and Mill and Division Streets would be required. Interior drainage and sanitary sewage would be handled by separate systems and pumping stations. Construction of the plan would require the taking of 14 acres of land and the moving of 2 small buildings.

In the River Street area above Maple Street, dike and flood wall protection would start at high ground near Maple Street, extend upstream about 600 feet along the landside of River Street, and terminate at high ground about 650 feet west of the Naugatuck River. Three stoplog structures and a small pumping station would be required. Construction would require the taking of about three acres of land.

b. East bank. Protection for the east bank area would be comprised of 5,050 feet of dike and 4,450 feet of flood wall and would extend from the vicinity of the Central Street bridge across Beaver Brook, downstream along the northerly bank of the brook about 2,500 feet to the main stem of the Naugatuck River, and upstream about 7,700 feet generally along the present and relocated river bank to the vicinity of the American Brass Company hydroelectric plant. Two pumping stations, for handling interior drainage, and five stoplog structures, for railroad and street crossings, would be required. About 1,600 feet of railroad spur track, and 350 feet of urban street would be relocated. The construction would require the taking of 39 acres of land and the acquisition of 7 buildings.

c. Channel modification. The plan for channel improvement in this report is based on hydraulic studies which determined the most practical channel capacity compatible with the available waterway at the several bridges in the reach. Work would consist of deepening the Naugatuck River channel from a point upstream of the American Brass Company private bridge to just below the Bridge Street bridge; widening on the west bank in the vicinity of the Maple Street bridge; and relocating and widening downstream of the railroad bridge. A stream deflector would be constructed on the center pier of the New Haven Railroad bridge to increase river flows through the northerly bridge opening and to minimize the build-up of silt deposits under the northerly span.

37. OTHER PLANS STUDIED

a. West bank. Three alternative alignments of the dike below Division Street, in addition to the plan described above, were considered: the first to provide local protection in Derby, including the Charlton Printing Company and a small area to the south, but excluding the drive-in theater; the second including only the printing company; and the third extending along Division Street, thereby excluding all property in Derby. All three plans were more costly than the selected plan, while realizing fewer benefits.

North of Division Street one alternative alignment, in addition to the plan described above, was considered. This alignment would turn inland just above Division Street, pass behind the Hershey Manufacturing Company, and run parallel to the New Haven Railroad tracks to tie into high ground at the same point as the most feasible plan. The cost of this plan is about the same as the selected plan but it would not realize the enhancement benefits to the large, vacant flood plain area protected by the selected plan. The benefit-cost ratio of this alternate is less than unity.

In the River Street area, two additional plans were studied: one to provide a wall along the riverside of River Street; the other to provide a wall along the face of the Ansonia Manufacturing Company buildings with window and door closures. The upstream end of the wall, in both plans, would tie into a dike north of the plant. The downstream end of the wall, in the first plan, would terminate at a sandbag closure at the junction of River and Maple Streets; in the second plan it would tie into a dike extending to the Maple Street road fill. The alignment along the riverside of River Street is more costly than the most feasible plan, while realizing essentially the same benefits. Local interests prefer the plan with the protection along the face of the Ansonia Manufacturing Company buildings - also more costly than the most feasible plan - and are willing to pay the additional cost for this plan.

b. East bank. The only feasible alternative plan on the east bank would substitute a pressure conduit for dikes and walls along the lower reach of Beaver Brook. This plan was found to be considerably more costly than the proposed plan while providing only about one percent more benefits.

38. HYDROLOGIC AND HYDRAULIC CONSIDERATIONS

The Ansonia-Derby local protection project is designed to protect against a flood of 75,000 c.f.s. at Ansonia, which represents the standard project flood reduced by authorized and recommended reservoirs.

39. DEGREE OF PROTECTION

The local protection projects described in this report would provide complete protection against the project design flood with a minimum freeboard of three feet.

40. PROVISIONS AGAINST ENCROACHMENT

Under existing legislation, the Water Resources Commission of the State of Connecticut has established river encroachment lines in Ansonia and Derby which will insure against future building of any obstruction or other encroachment that would tend to lessen or nullify the effectiveness of any local protection project.

SECTION XVI - ESTIMATES OF FIRST COSTS AND ANNUAL CHARGES

41. FIRST COSTS

Unit prices used in estimating construction costs are based on average bid prices for similar work in the same general region, adjusted to the January 1960 price level. Valuations of property are based on information from local officials and reflect values in recent sales in the area. All costs include an allowance for contingencies and for minor items of work which do not appear in the estimate. The estimated costs for engineering and overhead are based on knowledge of the site and experience on similar projects. A summary of first costs for the most feasible plan and the plan preferred by local interests is given in Table 5. A detailed breakdown of the costs of these plans is given in Appendix C.

42. ANNUAL CHARGES

Average annual costs, also summarized in Table 5, are based on interest rates of 2-1/2 percent for Federal costs, 3 percent for non-Federal costs and 6 percent for loss in productivity of land. Investment costs are amortized over the 50-year assumed life of the project. Allowances are made for costs of maintenance and operation and for interim replacement of equipment having an estimated life of less than 50 years. No loss of taxes is included as it is considered that the loss in taxable lands would be more than offset by the increase in the value of properties protected.

TABLE 5

SUMMARY OF FIRST COSTS AND ANNUAL CHARGES
ANSONIA-DERBY LOCAL PROTECTION
(Jan. 1960 Price Level)

	Most Feasible Plan	Plan Preferred by Local Interests
First Costs		
Lands and damages	$ 120,000	$ 120,000
Relocations	152,000	152,000
Channels and canals	431,000	431,000
Levees and flood walls	3,462,000	3,530,000
Pumping plants	793,000	793,000
Total direct cost	4,958,000	5,026,000
Preauthorization studies	20,000	20,000
Model study	50,000	50,000
Engineering and design	483,000	489,000
Supervision and administration	429,000	435,000
Total first cost	$5,940,000	$6,020,000
Annual Charges		
Interest	157,800	160,400
Amortization	61,000	61,700
Maintenance and operation	5,500	5,500
Interim replacements	5,200	5,200
Total financial annual charges	229,500	232,800
Loss of productivity of land	2,300	2,300
Total economic annual charges	$ 231,800	$ 235,100

SECTION XVII - ANNUAL BENEFITS

43. FLOOD PREVENTION BENEFITS

Flood damage prevention benefits for the Ansonia-Derby protection projects studied were derived by determining the difference between average annual losses of $288,000 remaining after discharge reductions by the authorized and recommended reservoirs and the average annual losses remaining after the addition of protective works. Supplementing the reservoir system, the proposed local protection project would provide complete protection against the loss potential from the standard project flood in the east bank area extending from north of the American Brass Company to the mouth of Beaver Brook and in two west bank areas - the River Street area and the Division Street area extending from the New Haven Railroad bridge to downstream of the Division Street bridge. Average annual flood damage prevention benefits accruing to the protective works are shown in Table 6 on page 29. Benefits would be the same for the most feasible plan and for the plan desired by local interests.

44. ENHANCEMENT BENEFITS

A survey of economic conditions and an analysis of flood losses and benefits were made in the areas considered for flood protection in order to determine the extent of enhancement or higher utilization of lands and buildings which could be expected as a result of project construction. This analysis revealed that substantial use of idle lands and vacant industrial and commercial space can be expected to follow the construction of the local protection project. The value of this enhancement represents additional benefits to the project.

A study of past, present, and future use of idle lands and vacant commercial and industrial space was made to obtain sufficient data for a sound economic analysis of potential enhancement. Valuable information was obtained from various responsible sources such as city officials, bankers, real estate brokers, industrial managers, merchants, and the Connecticut Development Commission. A recent study by the Economics Department of Yale University titled "The Economic Outlook for the Naugatuck and Farmington River Valleys" was also a source of valuable data.

The investigation revealed that a specific potential for enhancement exists in the project protection areas. The summary in Table 6 on page 29 lists the estimated average annual enhancement benefits.

45. INTANGIBLE BENEFITS

Improvement in the general welfare and security of the people of Ansonia and Derby, while not measurable in monetary terms, is nevertheless a highly significant benefit to be realized through construction of the local protection works. The economy of the region would benefit from revitalization of industry and commerce in the affected areas. In addition to the nearly complete reduction of the threat of loss of life and physical injury to residents in the flood area, construction of the projects would virtually eliminate from the protected areas most of the dangers accompanying a general flood, such as the widespread menace of polluted and disease-bearing floodwaters and hazards attendant upon emergency evacuation of whole areas.

Major metal industries which were important contributors to the national defense effort in World War II and during the Korean conflict, and which would be afforded protection by the projects, include the American Brass Company and the Farrel-Birmingham Company. The projects would aid in preserving and encouraging growth in these component parts of the national security.

TABLE 6

SUMMARY OF AVERAGE ANNUAL BENEFITS
(1960 Price Level)

Type	East Bank	West Bank	Total
Flood damage prevention	$147,000	$59,000	$206,000
Enhancement			
Land	8,300	28,600	36,900
Industrial space	29,300	-	29,300
Commercial space	17,800	-	17,800
Total Enhancement	55,400	28,600	84,000
Total Average Annual Benefits	$202,400	$87,600	$290,000

SECTION XVIII - PROJECT FORMULATION AND ECONOMIC JUSTIFICATION

46. GENERAL

Average annual benefits attributable to the most feasible plan, including both flood damage prevention and enhancement, exceed the annual costs by a ratio of 1.3 to 1. Benefits to the plan preferred by local interests exceed costs by a ratio of 1.2 to 1. The costs of protection for each bank compared with the benefits to be derived resulted in favorable benefit-cost ratios in all cases. For this latter comparison, one-half the cost of the channel work was added to the cost of protection for each bank to arrive at the total cost for each bank.

SECTION XIX - APPORTIONMENT OF COSTS AMONG INTERESTS

47. GENERAL

Since land enhancement or higher utilization benefits are expected to be realized in addition to the flood damage prevention benefits, local interests are required to contribute toward the construction cost of the project. The amount of the cash contribution is equal to 50 percent of the amount determined by applying to the total cost of the project, exclusive of preauthorization costs, the ratio of enhancement benefits to total benefits. Since local interests pay for all lands, damages, and relocations, currently estimated at $300,000, an appropriate reduction in their cash contribution is allowed. Local interests would also be required to pay the additional cost of the plan preferred by them over the cost of the plan found in this report to be the most feasible. Table 7 summarizes the division of costs between Federal and local interests for the two plans. These allocations indicate a total cash contribution by local interests of $770,000, equivalent to 13.7 percent of the construction cost, for the most feasible plan, and $850,000, equivalent to 14.9 percent of the construction cost, for the plan preferred by local interests.

TABLE 7

APPORTIONMENT OF COSTS SUMMARY
(Jan. 1960 Price Level)

	Most Feasible Plan	Plan Preferred by Local Interests
First Cost		
Federal	$4,870,000*	$4,870,000*
Non-Federal		
Lands, etc	300,000	300,000
Cash contribution	770,000	850,000
	1,070,000	1,150,000
Total	5,940,000	6,020,000
Annual Charges		
Federal	176,000	176,000
Non-Federal		
Financial	53,500	56,800
Economic	55,800	59,100
Total		
Financial	229,500	232,800
Economic	231,800	235,100
Annual Benefits		
Flood prevention	206,000	206,000
Enhancement	84,000	84,000
Total	$ 290,000	$ 290,000
Benefit-cost ratio	1.3	1.2

*Includes $20,000 preauthorization study costs.

SECTION XX - PROPOSED LOCAL COOPERATION

48. GENERAL

In accordance with Section 3 of the 1936 Flood Control Act, as amended, local interests would be required to provide, without cost to the United States, all lands, easements, and rights-of-way necessary for the construction and operation of the local protection project; hold and save the United States free from damages due to the construction works; and maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of the Army

Acquisition of lands required for spoil disposal areas, pumping stations, storage ponds, and collector ditch, or perpetual easements thereon, would also be the responsibility of local interests, as would the necessary modification and relocation of the municipal sewage system, minor utility relocations, and modification of two bridges, under the requirements of lands, easements, and rights-of-way.

Local interests would also be required to provide assurances that encroachment on the improved channels and on ponding areas will not be permitted and that, if ponding areas and/or capacities are impaired, substitute storage capacity or equivalent pumping capacity will be provided promptly without cost to the United States.

Local interests would further be required to contribute to the United States 14.9 percent of the construction cost due to enhancement benefits expected to be realized by the project and the additional cost of the plan preferred by them over the cost of the plan found to be most feasible. This cash contribution is currently estimated to be $850,000.

There is an intense desire for flood protection in the cities of Ansonia and Derby. State and city officials have indicated a willingness and ability fulfill the conditions of local cooperation.

SECTION XXI - COORDINATION WITH OTHER AGENCIES

49. GENERAL

Plans for local protection at Ansonia and Derby have been reviewed by Federal, State, and local agencies and industrial officials concerned, including the U. S. Fish and Wildlife Service, the Connecticut State Water Resources Commission, the Naugatuck Valley River Control Commission, the Mayors of Ansonia and Derby, the Ansonia Flood and Erosion Control Board and the Ansonia Redevelopment Agency. These agencies and individuals are all in favor of the proposed local protection project.

The Chief, Division of Technical Services, Bureau of Sports Fisheries, Fish and Wildlife Service of the Department of the Interior, has commented that it does not appear that the local protection works planned for the vicinity of Ansonia, Connecticut on the Naugatuck River would be in any way detrimental to fish or wildlife resources.

SECTION XXII - DISCUSSION

50. FLOOD PROBLEMS

Additional flood protection for the city of Ansonia and the contiguous area of Derby is urgently needed. Industrial, commercial, and residential properties have suffered widespread damage in six major floods in the past 30 years, resulting in disruption of the area economy. Even with the completion of the authorized Thomaston Reservoir and the construction of the other authorized and recommended reservoirs, a recurrence of the August 1955 flood would still produce losses of nearly $14 million along the Naugatuck River in the Ansonia-Derby area.

51. SOLUTIONS CONSIDERED

All practicable methods for solving the problem of flooding in the Ansonia-Derby area were considered, including construction of additional flood control reservoirs, channel widening and deepening, and construction of dikes and flood walls. Construction of sufficient upstream reservoirs to reduce flood stages to channel capacity in a project design flood would be economically infeasible. Channel improvement alone would neither solve the flood problem nor be economical. The only method found worthy of detailed study consisted of dikes and flood walls along both banks of the Naugatuck in combination with channel relocation and improvement.

52. SELECTION OF PLANS

Two alternative plans to protect the three areas subject to serious flooding were developed in detail: the most economically feasible plan and a slightly more expensive plan which is preferred by local interests. Both plans are economically justified and would realize the same benefits. Both plans are designed to protect the flooded areas from the standard project flood. Since local interests desire the more expensive plan and have indicated a willingness and ability to pay the added cost over that of the most economically feasible plan, the plan preferred by local interests is the one selected.

Additional information on recommended and alternative projects called for by Senate Resolution 148, 85th Congress, 1st Session, adopted 28 January 1958, is contained in Attachment II to this report.

SECTION XXIII - CONCLUSIONS AND RECOMMENDATIONS

53. CONCLUSIONS

A recurrence of the flood of August 1955 would produce major flood damages in the highly industrialized and urbanized area along the Naugatuck River in Ansonia and Derby. The area would still suffer residual damages approximating $14 million even after construction of all authorized and recommended reservoirs in the Naugatuck River Basin.

Protection can best be provided by the construction of a local protection project on both banks of the Naugatuck River, essentially as described in this report. The plan would afford a high degree of protection and is economically justified. The amount of damage to which the area is susceptible makes immediate construction of this project imperative.

54. RECOMMENDATIONS

It is recommended that the construction of a local protection project on the Naugatuck River in Ansonia and Derby, Connecticut be authorized essentially as described in this report with such modifications thereof as in the discretion of the Chief of Engineers may be advisable; at an estimated cost to the United States of $4,850,000 for construction; provided that, prior to construction, local interests give assurances satisfactory to the Secretary of the Army that they will (a) provide, without cost to the United States, all lands, easements, and rights-of-way necessary for the construction and operation of the project, including lands for spoil disposal areas, pumping stations and drainage systems; (b) hold and save the United States free from damages due to the construction works; (c) maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of the Army; (d) permit no encroachment on the improved channels or on the ponding areas and, if ponding areas and/or capacities are impaired, will provide substitute storage capacity or equivalent pumping capacity promptly without cost to the United States; and (e) contribute to the United States 14.9 percent of the construction cost, presently estimated at $850,000. The total first cost of the project is currently estimated at $6,000,000, exclusive of pre-authorization expenses. Costs for maintenance and operation, a local responsibility, are estimated at $10,700 annually, including $5,200 for major replacements.

ALDEN K. SIBLEY
Brigadier General, U. S. Army
Division Engineer

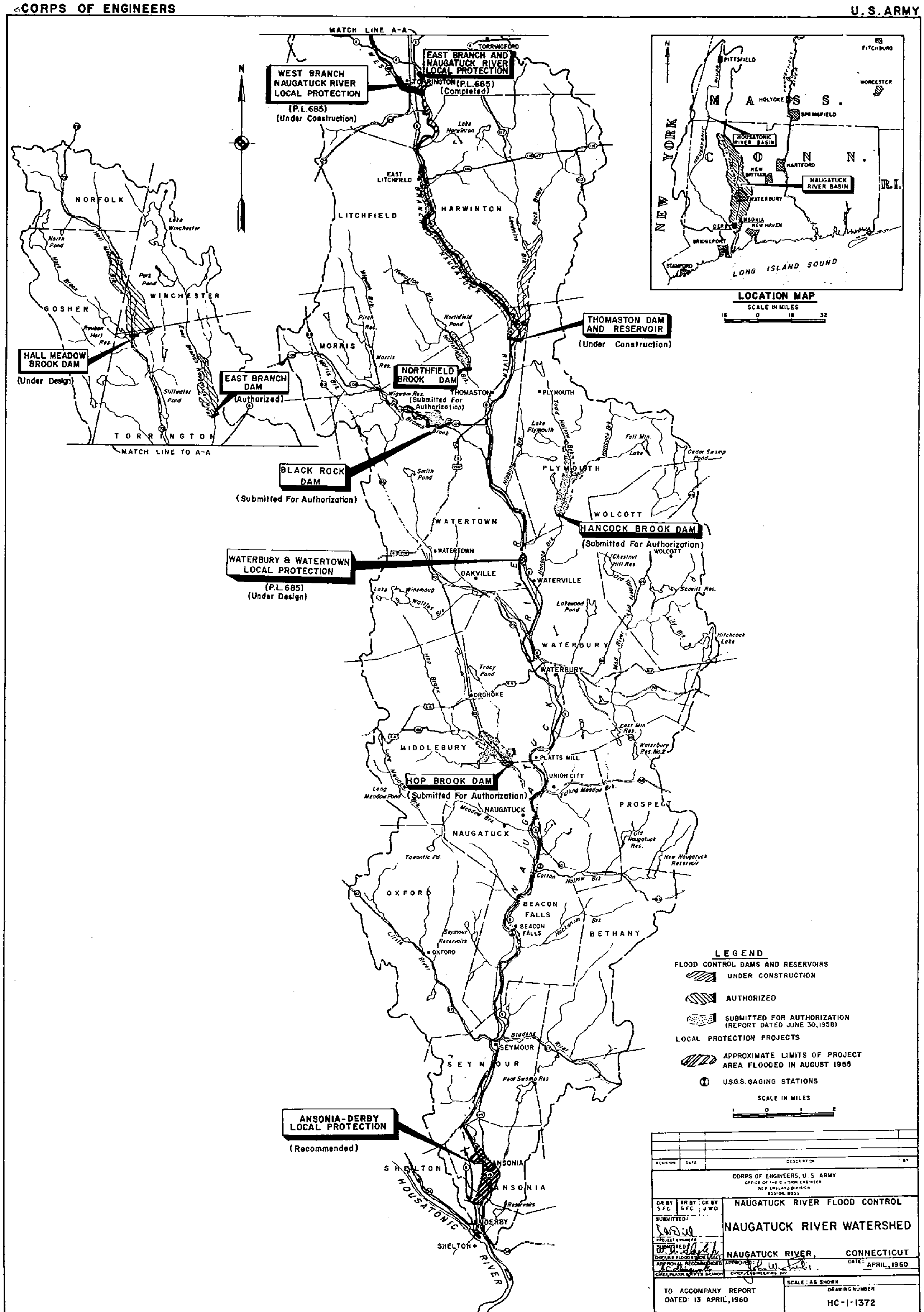

MATCH LINE A-A
EAST BRANCH AND NAUGATUCK RIVER LOCAL PROTECTION
(P.L.685)
(Completed)
WEST BRANCH NAUGATUCK RIVER LOCAL PROTECTION
(P.L.685)
(Under Construction)
TORRINGFORD
TORRINGTON
EAST LITCHFIELD
LITCHFIELD
HARWINTON
NORFOLK
WINCHESTER
GOSHEN
HALL MEADOW BROOK DAM
(Under Design)
EAST BRANCH DAM
(Authorized)
TORRINGTON
MATCH LINE TO A-A
MORRIS
NORTHFIELD BROOK DAM
THOMASTON DAM AND RESERVOIR
(Under Construction)
THOMASTON
PLYMOUTH
BLACK ROCK DAM
(Submitted For Authorization)
WATERTOWN
HANCOCK BROOK DAM
(Submitted For Authorization)
WOLCOTT
WATERBURY & WATERTOWN LOCAL PROTECTION
(P.L.685)
(Under Design)
OAKVILLE
WATERVILLE
WATERBURY
ORONOKE
MIDDLEBURY
PLATTS MILL
UNION CITY
HOP BROOK DAM
(Submitted For Authorization)
NAUGATUCK
PROSPECT
OXFORD
BEACON FALLS
BETHANY
SEYMOUR
ANSONIA-DERBY LOCAL PROTECTION
(Recommended)
SHELTON
ANSONIA
DERBY
HOUSATONIC RIVER
NAUGATUCK RIVER
LOCATION MAP
SCALE IN MILES
PITTSFIELD
FITCHBURG
WORCESTER
HOLYOKE
SPRINGFIELD
HARTFORD
NEW BRITAIN
WATERBURY
ANSONIA
NEW HAVEN
BRIDGEPORT
STAMFORD
LONG ISLAND SOUND
NEW YORK
MASS.
CONN.
R.I.
HOUSATONIC RIVER BASIN
NAUGATUCK RIVER BASIN
LEGEND
FLOOD CONTROL DAMS AND RESERVOIRS
UNDER CONSTRUCTION
AUTHORIZED
SUBMITTED FOR AUTHORIZATION (REPORT DATED JUNE 30, 1958)
LOCAL PROTECTION PROJECTS
APPROXIMATE LIMITS OF PROJECT AREA FLOODED IN AUGUST 1955
U.S.G.S. GAGING STATIONS
SCALE IN MILES
CORPS OF ENGINEERS, U.S. ARMY
NAUGATUCK RIVER FLOOD CONTROL
NAUGATUCK RIVER WATERSHED
NAUGATUCK RIVER, CONNECTICUT
DATE: APRIL, 1960
SCALE: AS SHOWN
TO ACCOMPANY REPORT DATED: 13 APRIL, 1960
HC-1-1372

PLATE NO. I

CORPS OF ENGINEERS U.S. ARMY

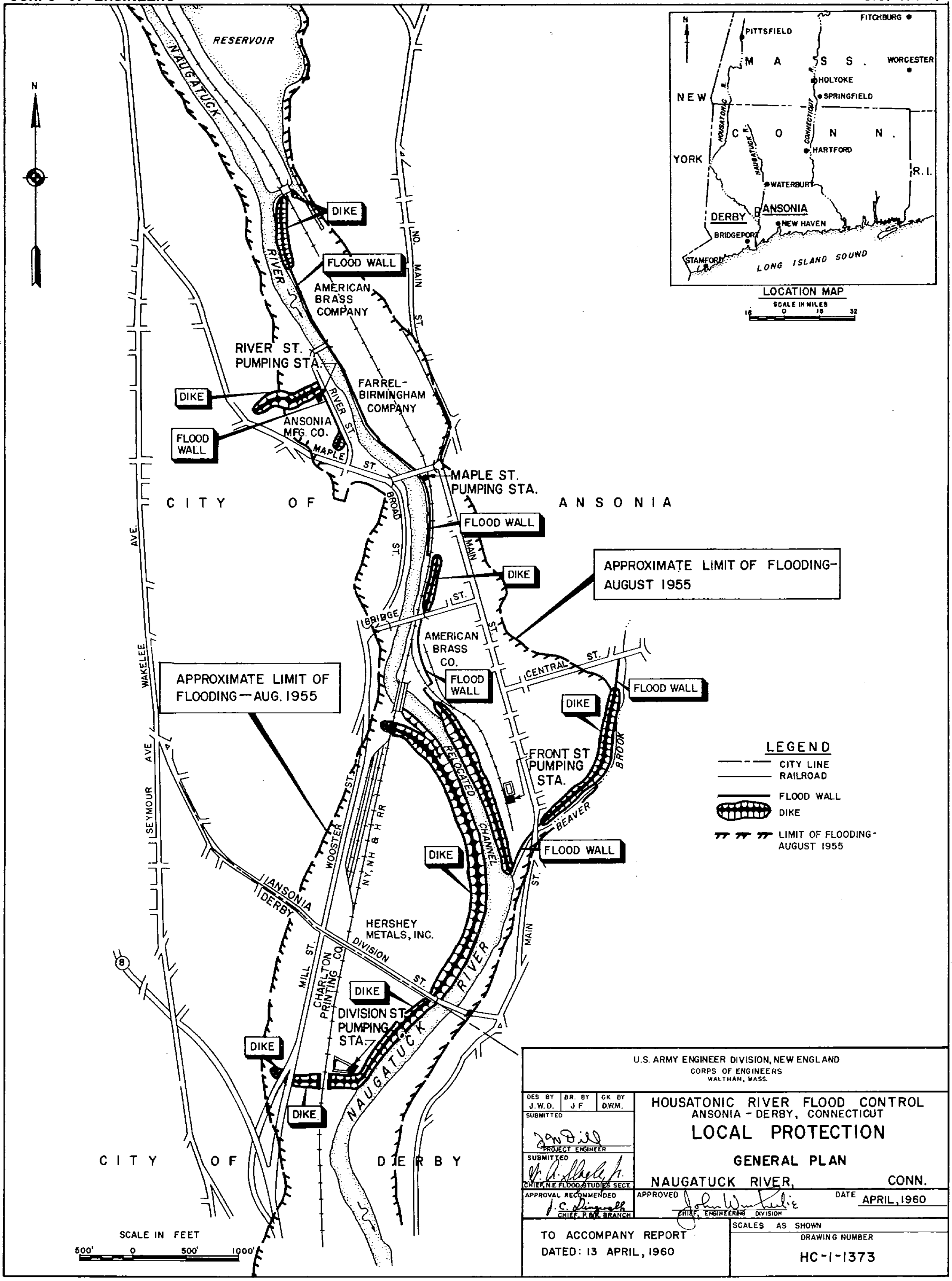


PLATE NO. 2

APPENDICES

APPENDIX A

HYDROLOGY AND HYDRAULICS

APPENDIX A

HYDROLOGY AND HYDRAULICS

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APPENDIX A

HYDROLOGY AND HYDRAULICS

1. INTRODUCTION

This appendix presents hydrologic and hydraulic data applicable to the development of a flood protection plan for the Ansonia-Derby area, Connecticut.

2. DESCRIPTION OF PROJECT AREA

a. Areas adjacent to Naugatuck River. The project area lies within the cities of Ansonia and Derby. The Naugatuck River, a major tributary in the Housatonic River system, bisects the city of Ansonia. The total drainage area of the Naugatuck River at the Division Street bridge is 311 square miles. Principal flood damage is centered in the industrial and commercial developments of Ansonia which extend along the east bank of the Naugatuck River. The proposed flood protection project would extend 2 miles along the Naugatuck River, beginning 1,300 feet below the Division Street bridge at river mile 1.0 in the city of Derby, and terminate near the American Brass Company hydroelectric plant at river mile 3.0 in the city of Ansonia. Through the project area, the Naugatuck River has an average channel slope of seven feet per mile. The river bed in this reach is unstable and deposits of sand and gravel continually shift about within the limits of the channel. Five bridges span the river and break the channel into relatively short and uniform reaches. Downstream from the New Haven railroad bridge, stages in the Naugatuck River are affected by tidal conditions in Long Island Sound. Only one minor tributary, Beaver Brook, enters the Naugatuck River in the Ansonia reach. A complete basin description of the Naugatuck River watershed is presented in the "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River, Connecticut," dated 30 June 1958.

b. Beaver Brook watershed. Beaver Brook, with a drainage area of 3.6 square miles, enters the Naugatuck River about midway between Division Street and the railroad bridge. Since a portion of the total drainage area is controlled by headwater reservoirs, a net area of 2.9 square miles is assumed to contribute to peak discharges at Central Street. The topography of the watershed is hilly and, therefore, conducive to rapid runoff. Beaver

Brook is 3 miles long and has a slope of 150 feet per mile. This relatively steep channel slope meets the Naugatuck River flood plain near Central Street, skirts the edge of the flood plain, following along the toe of a ridge; and then, for the last 0.2 mile of its course, cuts through the flood plain to the Naugatuck River.

3. CLIMATOLOGY

The climatology of the Naugatuck River watershed is published in the "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River, Connecticut," dated 3 June 1958.

4. RUNOFF AND STREAMFLOW DATA

a. Records. There are no published records of river stages or streamflows for the Naugatuck River at Ansonia or for Beaver Brook. Some unpublished records of river stages in the Naugatuck River during major flood periods were available from private sources. These were used to supplement and test the validity of computed river stages in the subject reach.

b. Runoff. The average annual runoff for the Naugatuck River basin, based on 33 years of record at the gaging station near Naugatuck, Conn., is about 470 c.f.s., equivalent to 26 inches from the tributary drainage area. This represents about 50 percent of the average annual precipitation, with one-third of the annual runoff normally occurring during the months of March and April.

5. FLOODS OF RECORD

a. General. Floods of record in the Naugatuck River basin are discussed in detail in the "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River, Connecticut," dated 30 June 1958. In the Beaver Brook watershed, the flood history indicates few, if any, instances of damage by flooding. However, this paucity of flood history may have resulted because the damage was relatively slight when compared with that caused by the Naugatuck River.

b. Frequency. The frequency or percent chance of occurrence of flood discharges was determined from records of all gaging stations in the Naugatuck and adjacent river basins. The frequency analyses were made in accordance with procedures described in Civil Works Engineer Bulletins 51-1 and 51-4. The method considers that the logarithmic value of annual peak flows are normally distributed, thereby permitting the application of standard statistical analysis. This enables the discharge-frequency curve to be defined by its mean values and standard deviation therefrom. The application to New England rivers is summarized in F. C. S. Memorandum No. 52 - General - 3, "Flood Frequency Studies in New England". Following the August and October floods of 1955, new frequency studies were initiated for all New England river basins. The mean and standard deviations for each gaged area were recomputed to include flow data for five additional years. Based on a regional analysis, the skew coefficient adopted for the Naugatuck River Basin was revised from 0.3 to 1.0. Natural and modified peak discharge frequency curves at Maple Street bridge, applicable to the Ansonia reach, are shown on Plate A-2.

6. ANALYSIS OF FLOODS

a. General. An analysis of all recent floods and the development of the standard project flood for the entire Naugatuck River Basin are discussed in the "Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River", dated 30 June 1958. The natural and modified hydrographs for the August 1955 flood and SPF at Ansonia, Connecticut are shown on Plate A-2.

b. Effect of tide and Housatonic River discharge.

(1) General. Stages in the Housatonic River at the mouth of the Naugatuck River are produced by the total concurrent flood discharges from the two rivers and the condition of the ocean tides in Long Island Sound. The floods of record demonstrate the effect which synchronization of these factors has upon water surface levels in the Ansonia-Derby area. Table A-1 shows a tabulation of the relationships between discharge, stage, and coastal tides during recent major floods. Plate A-3 shows the discharge rating curves for the Housatonic River at Shelton, Conn., below the confluence of the Naugatuck River for normal and abnormal tide conditions in Long Island Sound.

(2) Standard project flood stage. The water surface elevation in the tidal reach of the Naugatuck River below Division Street bridge, starting elevation for backwater computations, was computed and based upon the following assumptions:

(a) At Shelton, Conn., below the mouth of the Naugatuck River, the Housatonic River flow would have been 225,000 c.f.s. during the flood of August 1955 without Shepaug Reservoir.

(b) A coincident abnormal tide in Long Island Sound would produce an increase of two feet above the normal tide at Shelton.

(c) Head losses in the Naugatuck River between the mouth and Division Street would total five feet.

(d) At Division Street, the standard project flood peak modified by the comprehensive reservoir system (shown in Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River dated 30 June 1958) is 75,000 c.f.s.

The detailed analysis resulted in a starting elevation of 29.0 feet, m.s.l., as shown in Table A-2.

c. Standard project flood - Beaver Brook watershed. A standard project flood was developed for the Beaver Brook watershed, using standard project storm rainfall as described in Civil Engineer Bulletin No. 52-8 and a synthetic unit hydrograph derived from analyses of gaged areas within and adjacent to the Naugatuck River Basin. Plate A-2 shows the mass rainfall curves, unit hydrograph, and flood hydrograph for the Beaver Brook standard project flood.

(1) Standard project storm. The standard project storm was centered directly over the basin in order to produce maximum runoff in Beaver Brook. The standard project storm volume for the 48-hour storm period totaled 13.9 inches. Losses of 0.15 inches per 3-hour period were deducted from the rainfall, resulting in 11.5 inches of runoff.

(2) Unit hydrograph. A 3-hour unit hydrograph was derived for the 2.9 square mile drainage area, considered to be contributory to peak discharge at Central Street. This unit hydrograph was developed by employing basin characteristics and unit hydrograph constants adopted from similar gaged basins within the region. The unit hydrograph has a peak ordinate equal to 130 c.s.m.

(3) Standard project flood discharge. The standard project storm runoff applied to the unit hydrograph produces a peak discharge in Beaver Brook at Central Street of 3,180 c.f.s., equivalent to 1,100 c.s.m.

TABLE A-1

MAJOR FLOODS

HOUSATONIC AND NAUGATUCK RIVERS

Date of Flood	Water Surface Elevation at Division St. Br. (in ft., m.s.l.)	Contribution to Maximum Discharge in Housatonic River (in c.f.s.)			Water Surface Elevation at Shelton, Conn. (in ft., m.s.l.)	Concurrent Tide Condition at Derby, Conn.
		Naugatuck River	Housatonic River	Total		
Mar 1936	-	27,000	60,000	87,000	18.0	-
Sep 1938	21.3	30,000	60,000	90,000	19.6	Abnormal (1)
Dec 1948	18.8	32,000	50,000	82,000	17.2	Near normal high
Aug 1955	27.9	112,000	40,000	152,000	21.0	Normal high
Oct 1955	23.8	40,000	75,000	115,000	21.0	Abnormal (2)

(1) Abnormal high tide due to hurricane occurred 21 September at 8 PM. Maximum discharge occurred 22 September at 1 AM. Maximum stage recorded concurrently with abnormal high tide.

(2) Stages of Housatonic River at Devon, Conn., near the mouth remained two to three feet above predicted high tide throughout the flood. Maximum river stage occurred during abnormally high predicted low tide.

TABLE A-2

NAUGATUCK RIVER STANDARD PROJECT FLOOD
AT HOUSATONIC RIVER

Derby, Connecticut

Flood	River	Maximum Discharge (in c. f. s.)		Contribution to Maximum Combined flow at Shelton, Conn. (in c. f. s.)	Stage at Shelton Normal Tide (ft., m. s. l.)	Stage at Shelton Abnormal Tide (ft., m. s. l.)
Experienced Aug 1955	Naugatuck	112,000		105,000		
	Housatonic	70,000		55,000		
			Total	160,000	21.0	23.0
Aug 1955 without effect of Shepaug Reservoir	Naugatuck	112,000		105,000		
	Housatonic	130,000		120,000		
			Total	225,000	23.0	25.0
Standard Project Flood in Naugatuck River	Naugatuck	75,000 1/		70,000		
	Housatonic	130,000		120,000		
			Total	190,000	22.0	24.0

	Normal	Abnormal
Head loss, in feet, Naugatuck River, Division Street to Mouth	5.0	5.0
Water surface elevation below Division Street Bridge (ft., m. s. l.)	27.0	29.0

1/ Modified by comprehensive reservoir system. (Hall Meadow Brook, East Branch, Thomaston, Northfield Brook, Hancock Brook, Black Rock, and Hop Brook Reservoirs)

7. HYDRAULICS OF RECOMMENDED PLAN

a. General. The recommended flood protection plan for the Ansonia-Derby area consists of dikes and flood walls with the necessary companion structures. Miscellaneous structures included in the recommended plan are: stoplog structures, a stream deflector, interior drainage collection systems, pumping stations, and outfall conduits. Standardized methods used for similar projects in New England were employed in the hydraulic computations to insure that these structures would operate satisfactorily. Three feet of freeboard is generally provided for all dikes and flood walls, with additional height upstream of hydraulic control sections to insure protection from overtopping as the result of temporary lodging of debris in bridge members. Water surface profiles and dike and flood wall alignment are shown on Plate A-1 of this appendix.

b. Dikes and flood walls. The dikes and flood walls are designed to protect against the standard project flood modified by the proposed reservoir system (75,000 c.f.s.) recommended in the Interim Report on Review of Survey, Housatonic River Basin, Naugatuck River, dated 30 June 1958. Water surface profiles were determined by backwater computations, using Method 1 as described in the Engineering Manual for Civil Works, "Construction," Part CXIV, Chapter 9. Some areas, such as the New Haven Railroad bridge, will require special treatment as discussed below.

c. New Haven Railroad bridge. This railroad bridge presents a serious hydraulic problem. From cost and railroad alignment considerations, it is impractical to replace the existing structure. The bridge has very poor hydraulic characteristics due to its orientation and elevation with respect to the river. This bridge will be partially submerged and will present a serious restriction to flood discharge during major floods. The complexity of streamflow in the immediate vicinity of the bridge prevents accurate determination of the water surface under all expected conditions of discharge.

During past floods, deposits of sediment and debris have rendered the waterway opening under the northerly span almost useless. A stream deflector on the center pier of the present bridge has been included to aid in equalizing the existing openings and reduce deposition of gravel and heavy debris.

d. Superelevation of flood wall below Maple Street bridge. Superelevation of the east bank flood wall will be required around the bend immediately below Maple Street bridge to prevent overtopping. The superelevation will extend about 1,000 feet immediately downstream of Maple Street bridge and attain a maximum of about two feet.

e. Bridge losses. Bridge Street bridge has a high clearance and will pass the design flood without serious interference. The waterway areas of Division and Maple Streets are somewhat restrictive and will introduce some significant head losses, but improvements to channel alignment in the vicinity of the bridges will minimize the amount of lost head. Backwater studies indicate that individual head losses through these bridges will be between 1.0 and 1.5 feet during the modified standard project flood. The American Brass Company bridge will be inundated during a major flood and will produce some minor head loss.

f. Channel velocities. The average channel velocity will vary quite widely from section to section because of backwater effects. Minimum average velocities of about 6.5 to 7.0 f.p.s. will occur in areas affected by tidewater just downstream of the railroad bridge while maximum velocities of about 16 f.p.s. will occur at Maple Street bridge. In the greater portion of the reach, the average channel velocity will be between 8 and 12 f.p.s.

g. Hydraulic model study. Consideration will be given to a hydraulic model study of the proposed protection measures and improvements to assure adequate design. There are several features of the project, notably at bridges and bends, where hydraulic losses cannot be determined accurately. The effectiveness of the stream deflector at the railroad bridge would also be checked by model and modified, if necessary, to obtain best results. Present grades of dikes and walls are tentative and are subject to revisions based on model tests.

h. Interior drainage.

(1) General. The interior drainage areas total about 650 acres, of which about 70 percent is developed for residential and commercial use and about 30 percent for industrial use. The topography of the residential areas is hilly, while that in other areas is flat or gently sloping. The storm drainage system, which is essentially separate from the sanitary system, includes numerous outfalls

to the Naugatuck River. The existing storm drainage system is designed for a runoff of from 0.25 to 1.0 inch per hour per acre. Sanitary sewage is collected and discharged through a single 24-inch outfall into the river below the Division Street bridge.

(2) Interior drainage plan. The development of the proposed interior drainage plan, shown on Plate A-6, was accomplished by an Associate Professor of Civil Engineering employed by NED on a temporary basis. This plan is described in detail in Attachment I to this Appendix and was largely completed prior to receipt of recent Weather Bureau data on rainfall intensity - frequency relationships. Using design criteria set forth in the Preliminary Manuscript of Part CXIV, Chapter 10, "Interior Drainage of Leveed Urban Areas," of the Engineering Manual and rainfall values derived from United States Weather Bureau Technical Paper No. 29, "Rainfall Intensity-Frequency Regime," the design capacities of the pumping stations and gravity outfall structures, as proposed in Attachment I, have been reviewed and found to be satisfactory. In general, the review indicated that hydrologic criteria used in the preliminary design was slightly more severe than might be adopted in final design. Hence, the present estimated cost of drainage structures is considered adequately conservative to cover all costs and contingencies that might arise in final design. The general plan is briefly described in the following paragraphs:

(a) Area 1, a largely residential development located on the east bank, consists of 68 acres. The runoff from this area will discharge through a gravity system into the present process water canal and thence into the Naugatuck River at the American Brass Company hydroelectric station. The weir crest in the wasteway of the canal is at elevation 51.06 feet, m.s.l. Discharge rating and stage-frequency curves for the Naugatuck River at station 55+50 are shown on Plates A-4 and A-5, respectively.

(b) Area 2, consisting of the plant areas of the American Brass and Farrel-Birmingham companies, has a drainage area of 50 acres. The runoff from this area is collected and discharged either by the Maple Street pumping station during periods of high river stage or by gravity through the station outfall. Discharge rating and stage-frequency curves for the Naugatuck River at point of discharge, station 125+50, are shown on Plates A-4 and A-5, respectively.

(c) Area 3 comprises 165 acres of residential development on the east bank above Area 2. The runoff from this area will be collected and discharged through a pressure conduit which will outfall near the American Brass Company private bridge, station 139+00 on the Naugatuck River. Discharge rating and stage-frequency curves for the Naugatuck River at Station 139+00 are shown on Plates A-4 and A-5, respectively.

(d) Area 4 consists of 125 acres, the runoff of which will be discharged either by the Canal Street pumping station into Beaver Brook during periods of high river stage or by gravity into the Naugatuck River at several outfall locations. This east bank area includes the main commercial center of Ansonia and the adjacent residential sections. Hydrologic and hydraulic characteristics of the Naugatuck River at the point of discharge are assumed to be represented by the discharge rating and stage-frequency curves at station 85+00, which are shown on Plates A-4 and A-5, respectively.

(e) Area 5 is an industrial development on the west bank, comprising mainly the Ansonia Manufacturing Company plant. Runoff from this area will be discharged into the river near station 139+00 either by the River Street pumping station or by gravity through the station outfall. Discharge rating and stage-frequency curves for the Naugatuck River at station 139+00 are shown on Plates A-4 and A-5, respectively.

(f) Area 6, comprising about 40 acres of commercial and residential development above Area 5, normally drains into the municipal storm sewer system outfalling into the river just upstream of the New Haven railroad bridge. During periods of high stage in the Naugatuck River or when the municipal system becomes surcharged by intense rainfall, the runoff from this west bank sub-area will be discharged by a pressure conduit into an existing watercourse bordering the American Brass Company parking area north of the proposed dike. Discharge rating and stage-frequency curves for the Naugatuck River at station 139+ 00, considered representative for the point of discharge, are shown on Plates A-4 and A-5, respectively.

(g) Area 7 comprises 188 acres of industrial, commercial, and residential development on the west bank in the cities of Ansonia and Derby. The runoff from the area will be collected and discharged either by the Division Street pumping station or by gravity through the station outfall. Discharge rating and stage-frequency curves for the Naugatuck River at station 57+00, the point of discharge, are shown on Plates A-4 and A-5, respectively.

i. Beaver Brook Channel. Portions of the existing Beaver Brook channel will require excavation and realignment to insure against flooding behind the line of protection from overflow of Beaver Brook. The channel will be designed to have adequate capacity to discharge runoff from Beaver Brook watershed under variable conditions of flow and tailwater, namely:

1. The maximum rate of discharge in Beaver Brook (3,180 c.f.s.) produced by the standard project storm centered over Beaver Brook watershed, with concurrent negligible tailwater effect from the Naugatuck River.

2. A peak discharge in Beaver Brook of 2,250 c.f.s. produced by the standard project flood in the lower Naugatuck River Basin coincident with a Naugatuck River discharge of 45,000 c.f.s.

3. 1,000 c.f.s. discharge in Beaver Brook assumed coincident with the peak discharge in the Naugatuck River (75,000 c.f.s.) produced by the same standard project flood as in 2.

Computations indicate that the governing design criteria of the channel under the conditions stated above would be:

1. The flow, 3,180 c.f.s., would be supercritical above station 161+00 and at normal depth generally.

2. 2,250 c.f.s. flow would not produce critical design conditions.

3. Below station 161+00 backwater from the standard project flood in the Naugatuck River plus the concurrent flow of 1,000 c.f.s. in Beaver Brook would produce maximum water surface elevations.

8. ALTERNATIVE PLAN FOR BEAVER BROOK

An alternative plan to use a pressure conduit and a concrete lined chute for Beaver Brook was investigated but found to be more costly than the proposed plan.

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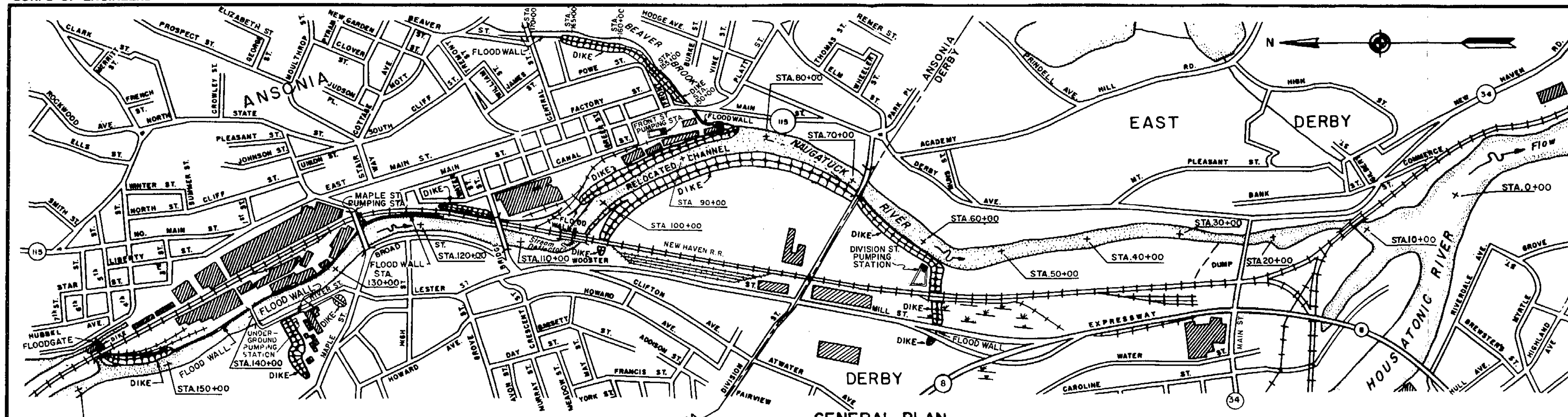
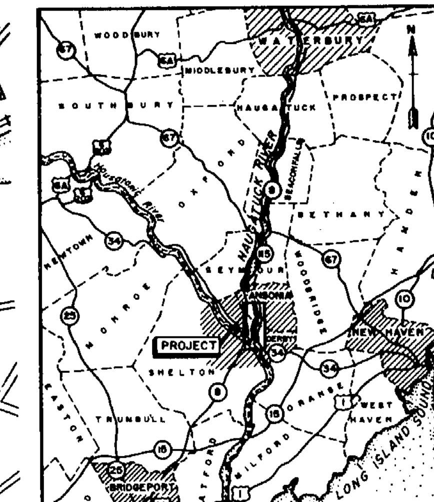
U. S. ARMY



GENERAL PLAN

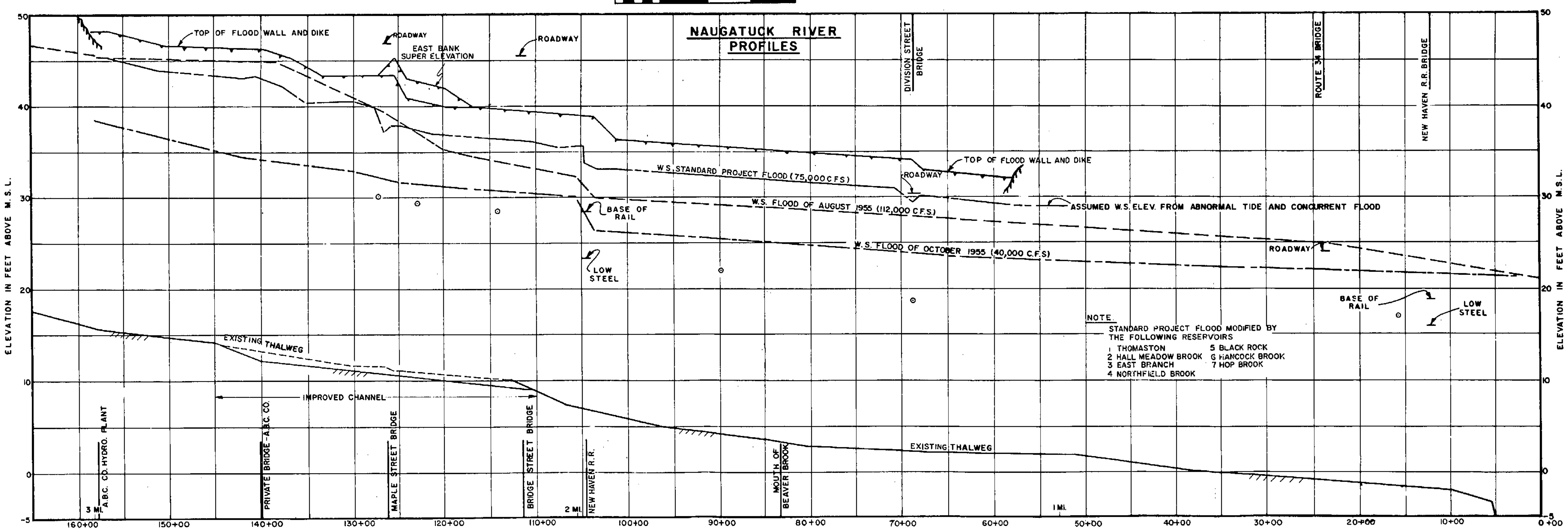
SCALE IN FEET

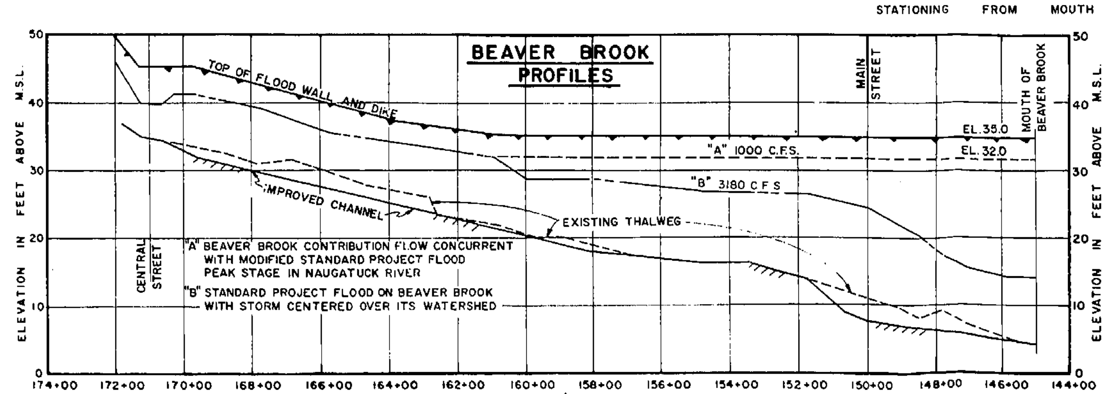
500 0 500 1000 1500



VICINITY MAP

SCALE IN MILES





LEGEND

- TOP OF DIKE AND / OR FLOOD WALL
- MODIFIED STANDARD PROJECT FLOOD PROFILE
- FLOOD OF AUGUST 1955 PROFILE
- FLOOD OF OCTOBER 1955 PROFILE
- FLOOD OF DECEMBER 1948 HIGH WATER MARKS
- THALWEG

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY R.W.M. | TR. BY | CK. BY E.P.S.

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY CONNECTICUT

LOCAL PROTECTION
PLAN & PROFILES
NAUGATUCK RIVER, CONN.

SUBMITTED BY: CHIEF, HYD. & HYD.
APPROVED: CHIEF, PLANN. & REP'TS BRANCH
APPROVED: CHIEF, ENGINEERING DIVISION

DATE: APRIL 1960

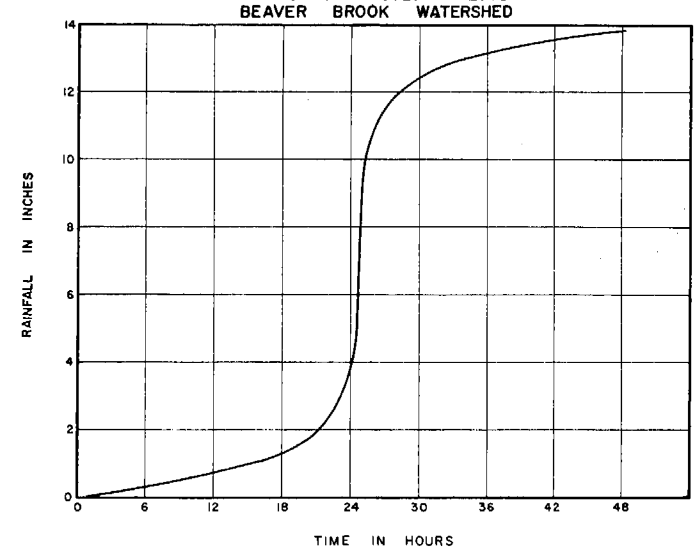
SCALE: AS SHOWN

TO ACCOMPANY REPORT DATED: 13 APRIL 1960

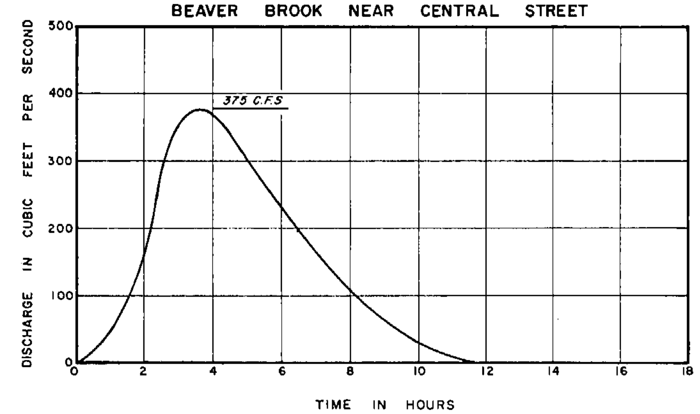
DRAWING NUMBER HC-1-1381

PLATE NO. A-1

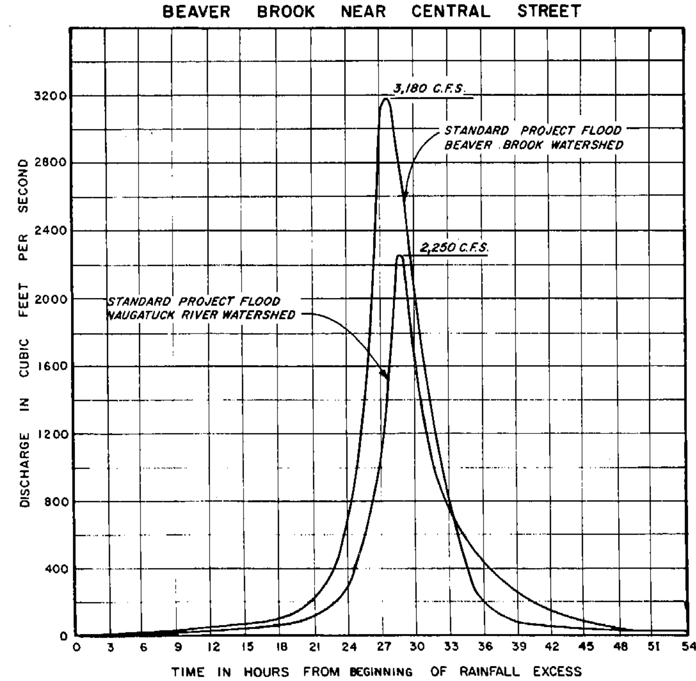
MASS CURVE OF RAINFALL
STANDARD PROJECT FLOOD
BEAVER BROOK WATERSHED



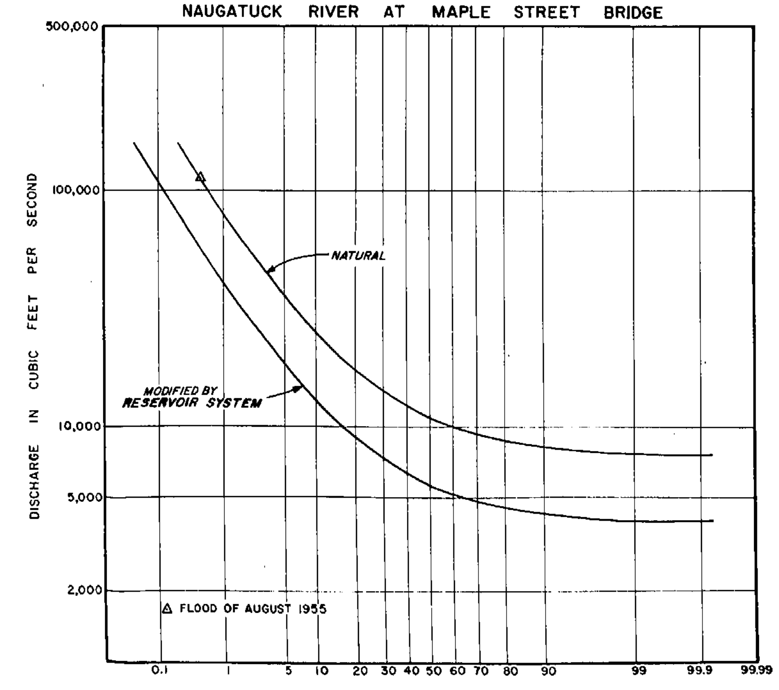
3 – HOUR UNIT HYDROGRAPH
BEAVER BROOK NEAR CENTRAL STREET



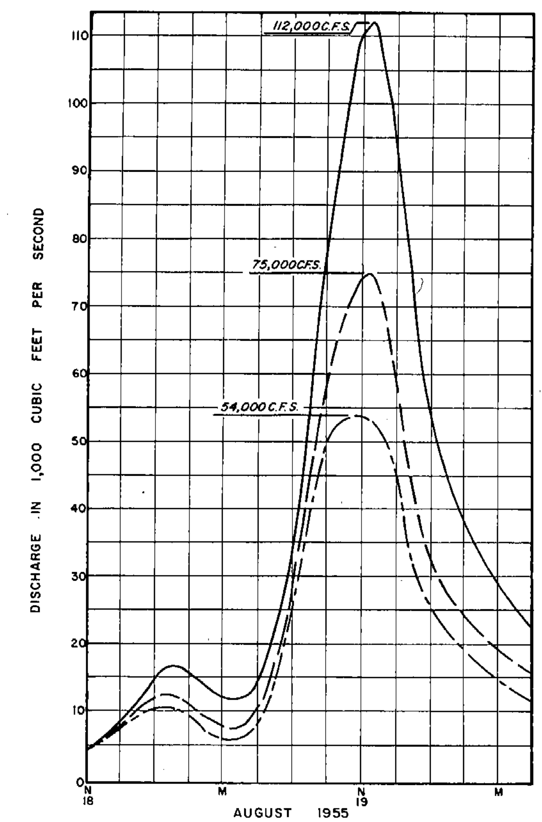
DISCHARGE HYDROGRAPHS
BEAVER BROOK NEAR CENTRAL STREET



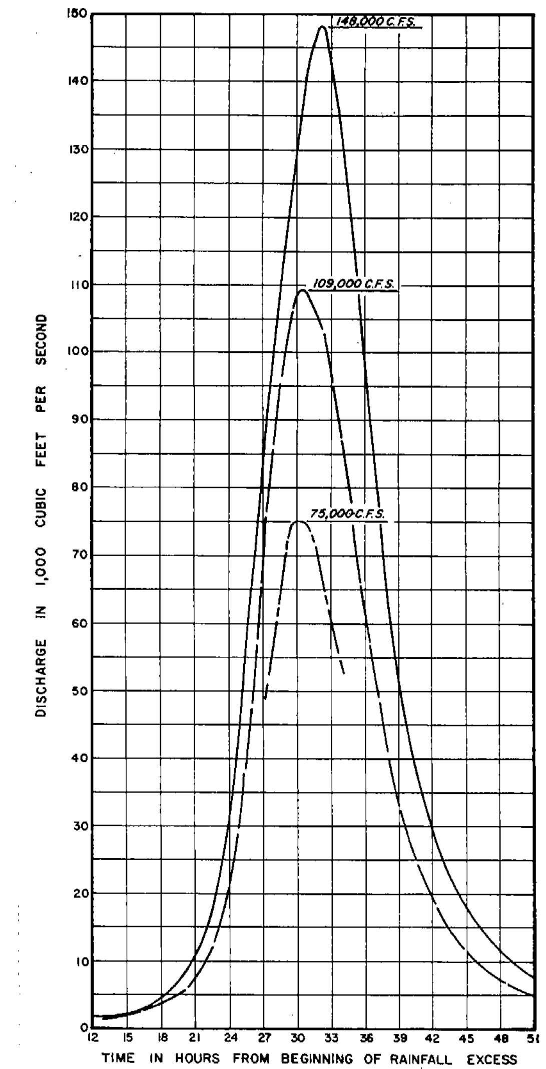
DISCHARGE FREQUENCY CURVES
NAUGATUCK RIVER AT MAPLE STREET BRIDGE



DISCHARGE HYDROGRAPHS
NAUGATUCK RIVER AT ANSONIA, CONN.



FLOOD OF AUGUST 1955



STANDARD PROJECT FLOOD

LEGEND

——— NATURAL
— — — MODIFIED BY THOMASTON RESERVOIR (ALSO APPLICABLE FOR THOMASTON, HALL MEADOW BROOK AND EAST BRANCH RESERVOIR)
—·—·— MODIFIED BY COMPREHENSIVE SYSTEM OF RESERVOIRS.
THOMASTON RESERVOIR (PLUS HALL MEADOW BROOK AND EAST BRANCH RESERVOIRS)
NORTHFIELD BROOK RESERVOIR
BLACK ROCK RESERVOIR
HOP BROOK RESERVOIR
HANCOCK BROOK RESERVOIR

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

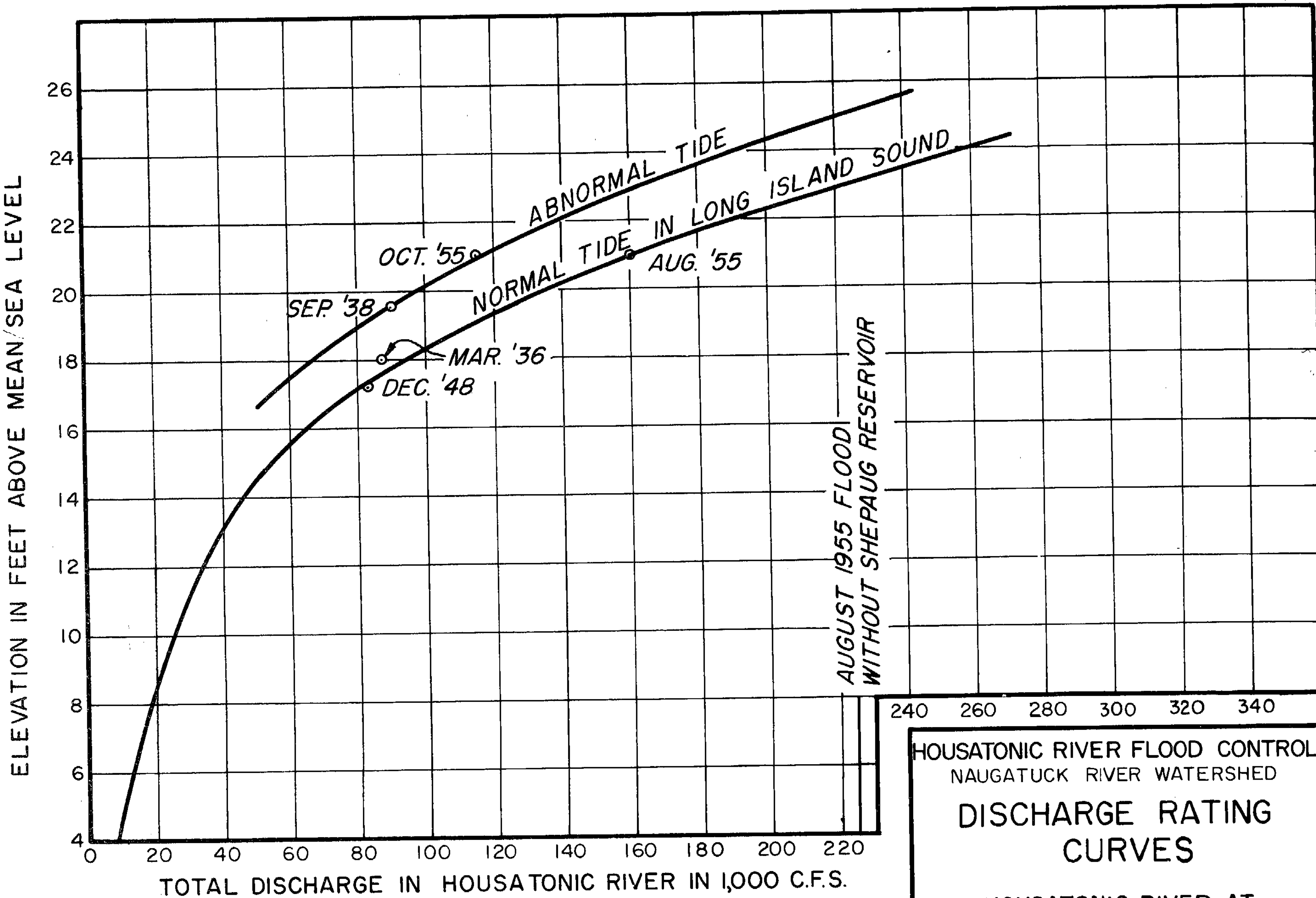
DR. BY	TR. BY	CK. BY
E.P.S.	M.W.B.	E.P.S.

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
HYDROLOGIC DATA
NAUGATUCK RIVER, CONN.

ENGINEER
SUBMITTED BY:
CHIEF, HYD. & HYD.
APPROVED:
CHIEF, PLANN. & REP'TS BRANCH
APPROVED:
CHIEF ENGINEERING DIVISION
DATE: APRIL 1960
SCALE: AS SHOWN
DRAWING NUMBER
HC-I-1382

TO ACCOMPANY REPORT
DATED: 13 APRIL 1960

PLATE NO. A-2



HOUSATONIC RIVER FLOOD CONTROL
NAUGATUCK RIVER WATERSHED

DISCHARGE RATING CURVES

HOUSATONIC RIVER AT SHELTON, CONN.

PLATE NO. A-3

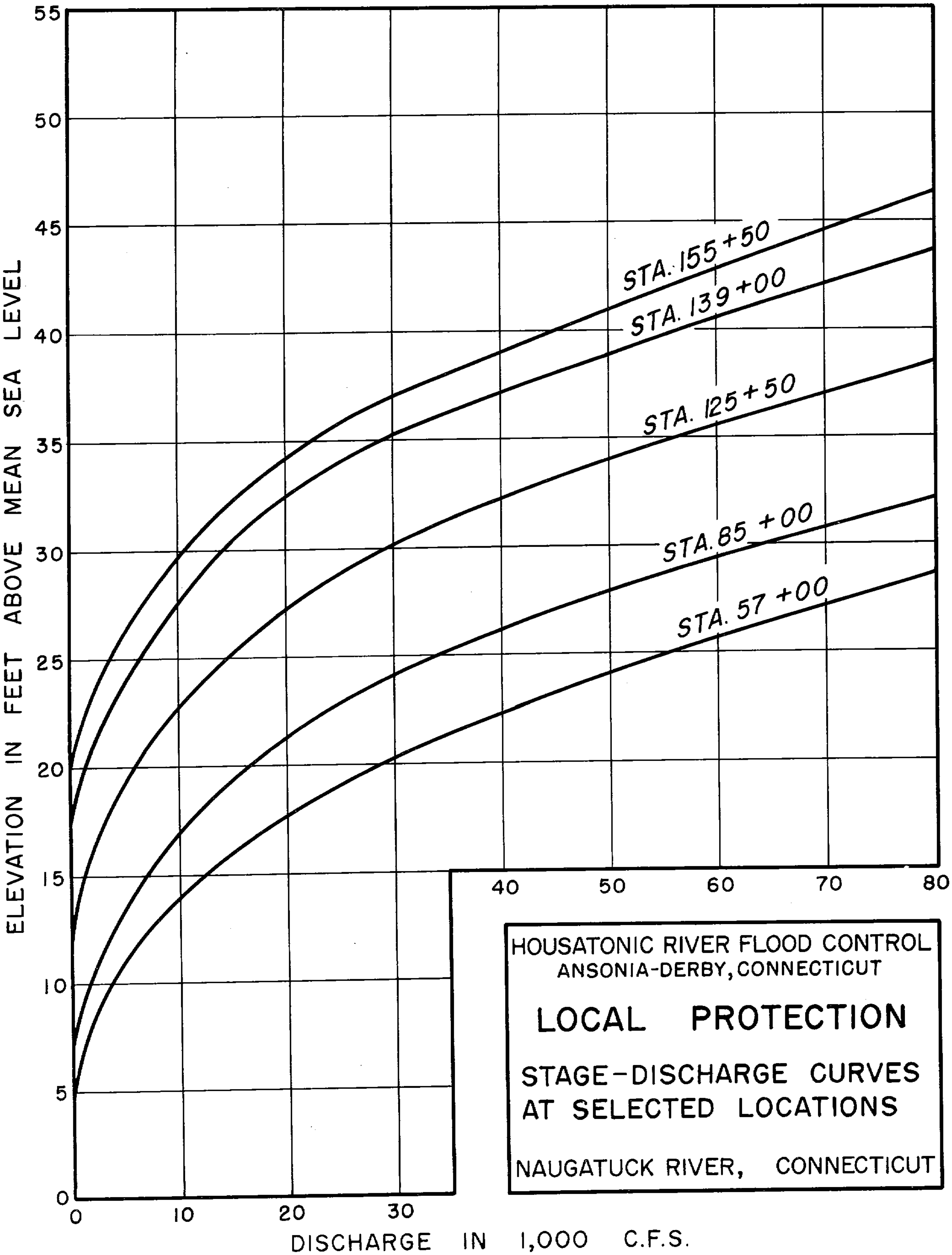

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE-DISCHARGE CURVES
AT SELECTED LOCATIONS
NAUGATUCK RIVER, CONNECTICUT
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
DISCHARGE IN 1,000 C.F.S.
STA. 155+50
STA. 139+00
STA. 125+50
STA. 85+00
STA. 57+00
0
5
10
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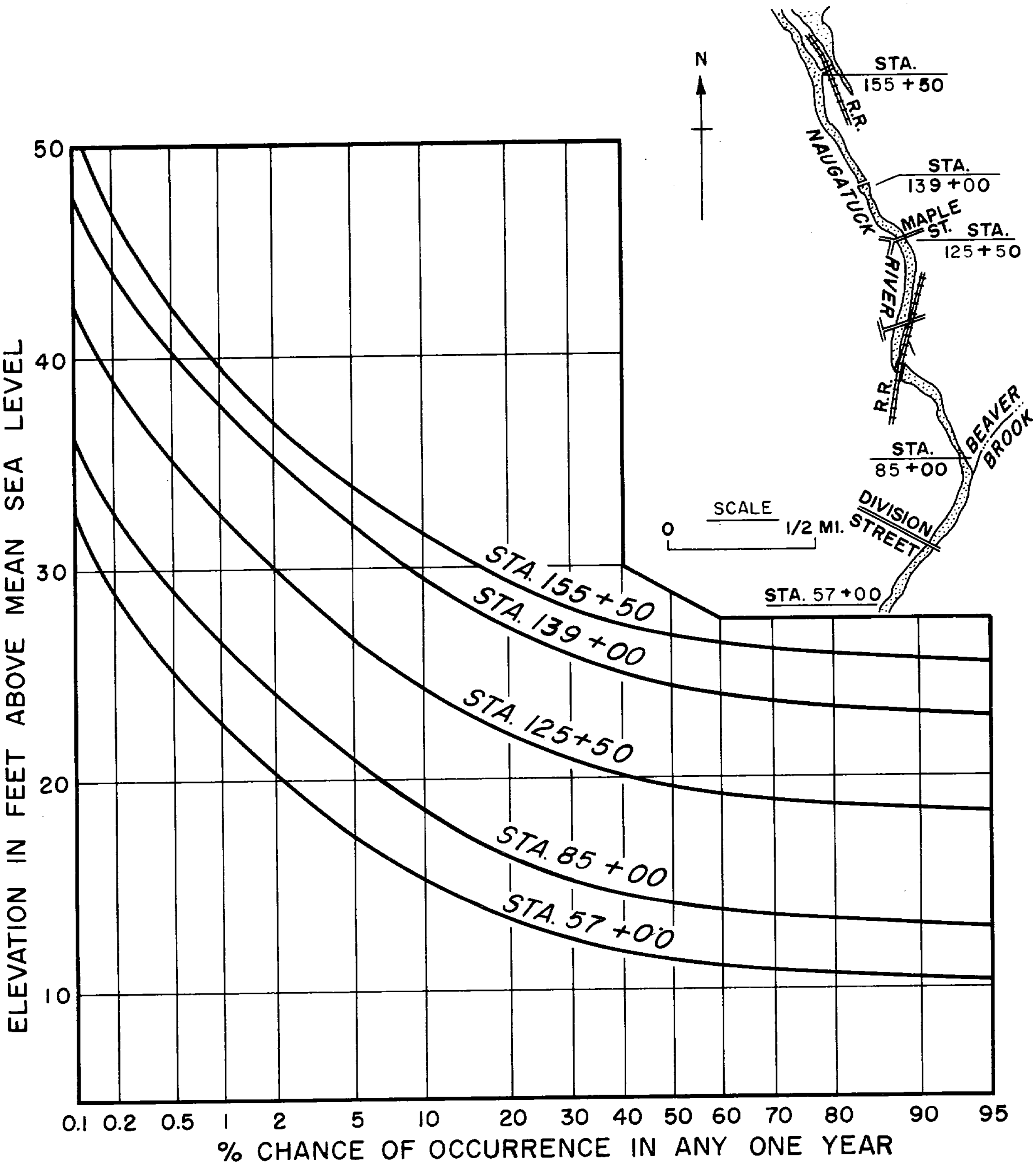
PLATE NO. A-4



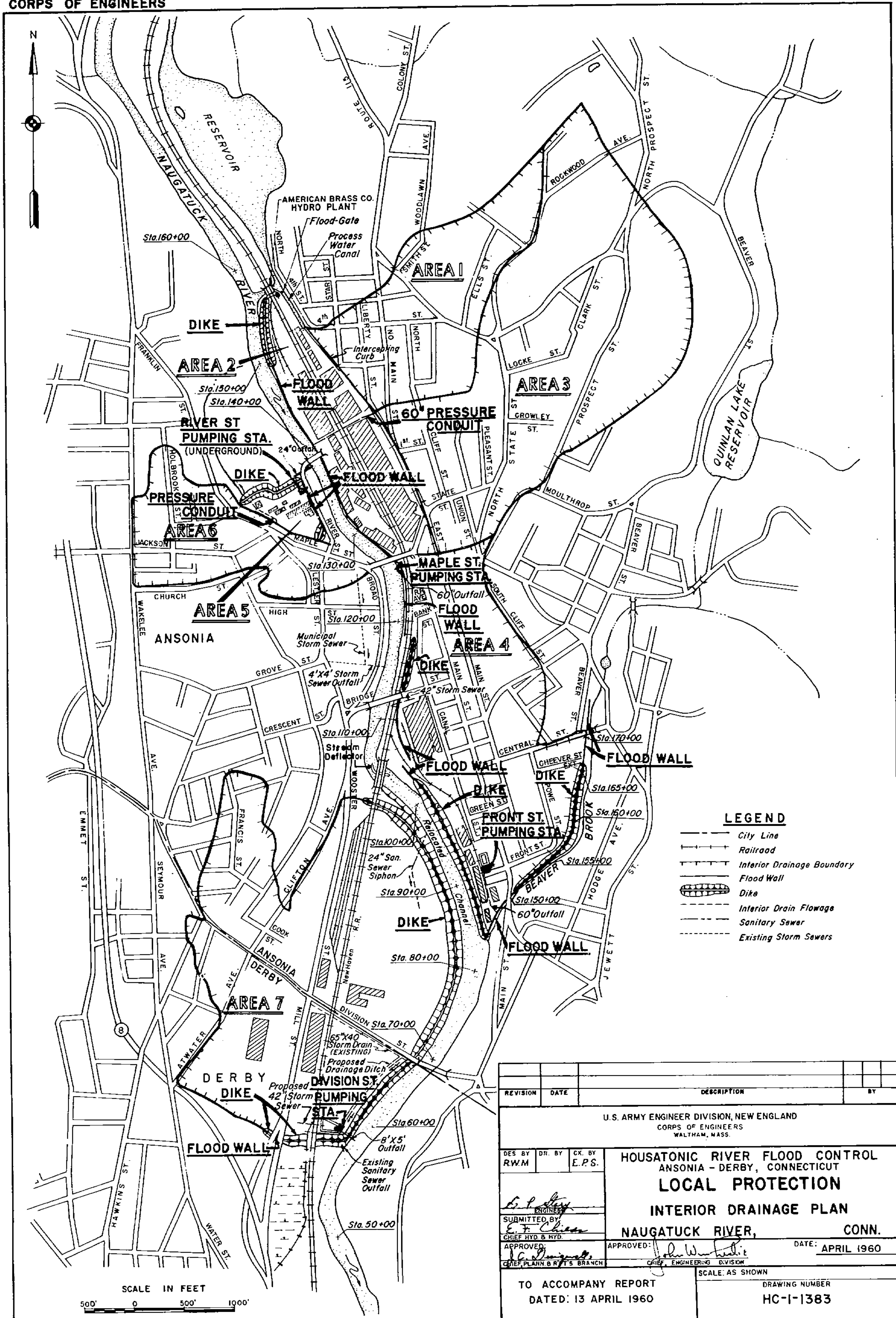
HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT

LOCAL PROTECTION

STAGE - FREQUENCY CURVES AT SELECTED LOCATIONS

NAUGATUCK RIVER, CONNECTICUT

PLATE NO. A-5



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HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT

LOCAL PROTECTION

INTERIOR DRAINAGE PLAN

NAUGATUCK RIVER, CONN.

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ATTACHMENT I

INTERIOR DRAINAGE REPORT

For

ANSONIA - DERBY, CONNECTICUT

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ATTACHMENT I

INTERIOR DRAINAGE REPORT

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ATTACHMENT I

1. DESCRIPTION OF THE PROBLEMS

a. Drainage areas and hydrology.

(1) Area 1.

(a) This area, shown on Plate A-6, is located northeast of the main business district of Ansonia on the east bank of the Naugatuck River at the upstream end of the protective works. The watershed of approximately 68 acres drains into the open canal between the American Brass Company plant and the powerhouse tailrace. The area is long and narrow in shape, having a length of 3,000 feet and a maximum width of 1,300 feet. The terrain is steep, varying in elevation from 300 feet, m.s.l. down to 50 feet, m.s.l. in a distance of approximately 3,000 feet. A network of right angle streets provides natural watercourses for overland flow. Most of the area is covered with a medium concentration of dwelling units with some small business establishments scattered throughout. The upper reaches of the area are at present undeveloped, but there are signs that development is under way. These undeveloped areas comprise open fields and light woods.

(b) Discharge from the watershed will be intercepted positively and conducted to the canal inside the protective works. Once in the canal the water will move upstream through the line of protection into the hydro plant tailrace, and then into the river. Flood gates will be constructed across the canal at the line of protection to prevent flood flows from entering the protected area while permitting reverse flow to occur, thus draining the upstream interior area.

(2) Area 2. This area is located on the flood plain within the protective works upstream of the Maple Street bridge. It is bounded by the Naugatuck River; the American Brass Company Canal; Liberty, North Main and Maple Streets. It comprises the plant areas of the American Brass and the Farrel-Birmingham companies upstream of Maple Street and totals about 50 acres. The area is long and narrow in shape, having a width of about 800 feet; the longest distance traversed by runoff is approximately 3,000 feet. The maximum difference in elevation in the area is about 20 feet with most of this difference occurring inland adjacent

to the streets. The remainder of the area is rather flat but with a slight pitch toward the river and Maple Street. Much of the terrain is covered with a heavy concentration of industrial buildings; the remainder is given over to paved or hard packed gravel storage yards and driveways. A main line track of the New Haven Railroad bisects the area.

(3) Area 3. This area is located northeast of the main business district of Ansonia on the east bank of the Naugatuck River. It is uphill of the Farrel-Birmingham industrial area. Runoff from this watershed would normally be included in those areas served by pumping stations. For reasons of economy, this area has been separated and will be served by a gravity force main discharging through the grounds of the American Brass Company. Most of the area discharges into a natural valley so that State Street becomes a watercourse conducting overland flow into the main business district. Some of the area discharges directly into the river. The area is shown on Plate A-6 and drains approximately 165 acres. The area is long and narrow having a generally curved shape with a maximum length of watercourse of about 5,500 feet and a maximum width of about 2,000 feet. Elevations range from 340 feet m.s.l. to 80 feet m.s.l. The upper reaches of the area drain into a natural valley in the vicinity of Clarke Street. From here it crosses North State Street between Crowley and Locke Streets to Pleasant Street. It follows close to Pleasant Street, thence to State Street, and then down State Street to Maple Street. Most of the area has a medium concentration of dwellings with development under way in the areas still open.

(4) Area 4. The watershed is located on the east bank of the Naugatuck River between Maple Street and Beaver Brook. It includes the main business district, the combined residential and business districts downstream, and some of the adjacent inland hills. Most of the area is a relatively flat flood plain except for the inland hills just east of the business district. A substantial area has been excluded from the existing drainage area on the assumption that the rebuilt Central Street Bridge will intercept the runoff. The watershed, shown on Plate A-6, has an area of approximately 125 acres and is long and narrow with a maximum length of water travel of about 4,000 feet. Elevations vary from 120 feet, m.s.l. to 20 feet, m.s.l., with the maximum change occurring in the outermost reaches of the area. The area

has a heavy concentration of commercial establishments, industrial establishments, and dwellings. There are several large paved parking lots and possibilities for more. Runoff from the watershed will be discharged into Beaver Brook by gravity or by pumping.

(5) Area 5. This area is located on the west bank of the Naugatuck River between River and Maple Streets as shown on Plate A-6. It includes the grounds of the Ansonia Manufacturing Company, is triangular in shape and has an area of approximately 8 acres, if provision is made to exclude overland flow into the area from other areas. The maximum distance for travel of water is about 700 feet and the maximum difference in elevation is about 30 feet, with most of the drop in elevation occurring near the outer landside boundary. For the most part the area is open; however, there are several dwellings near the boundaries of the area and one industrial establishment including several buildings. The remainder of the area is open woods or unpaved gravel parking lot. The parking lot, drives, and storage areas around the buildings are flat.

(6) Area 6. This area consists of 40 acres and is located to the rear of the Ansonia Manufacturing Company, uphill from Area 5. It is roughly triangular in shape and has a moderately steep slope towards the Naugatuck River. The elevation varies from about 40 feet, m. s. l. to 140 feet, m. s. l. in a length of 2, 600 feet. Drainage is provided by a municipal storm system which discharges into the river just upstream of the Bridge Street bridge. The area is highly developed, consisting of small commercial establishments and residences.

(7) Area 7. This area is located on the west bank of the Naugatuck River behind the downstream section of the protective works. It includes land in both the cities of Ansonia and Derby. The boundaries of the watershed are shown on Plate A-6. The area is irregular in shape and topography. The area can be divided into two topographic sections. The first section, between the New Haven Railroad tracks and the line of protection is flat with a maximum difference of 20 feet in elevation with many depressions marking the surface. The second section, west of the railroad tracks, is on a side hill with elevations varying from 160 feet, m. s. l. to 40 feet, m. s. l. The higher section slopes gently to moderately toward the river, becoming very steep

along the new Connecticut State Highway #8, also known as the Ansonia Connector. The total area, covering 188 acres, has an average width of about 3,000 feet and the average length about the same. The maximum distance traveled by water running off from the area is about 4,500 feet. It has been assumed that the protective works can be aligned so as to exclude the runoff from other areas which may be intercepted by the cut made for the Route #8 highway. West of the Ansonia Connector, the area is mostly populated by a medium density of dwellings and commercial establishments. There are also some open areas. East of the Ansonia Connector the area is largely undeveloped except for a few industrial establishments located adjacent to the Ansonia Connector and the railroad. Some of the area is utilized by an open air theater.

b. Sanitary sewerage systems.

(1) General. The existing sanitary sewer locations and the delineation of the areas serviced along with the proposed extensions to the systems, the additional areas to be serviced, and the proposed extension dates are shown on a plan entitled "Ansonia, Derby, and Shelton Connecticut, Present Sewerage Systems and Proposed Trunk Sewers", dated August 1, 1933. Scale 1 inch equals 400 feet. The plans show the obvious division into two sectors, one of the east bank, the other of the west bank. It also notes the connections between the two sectors, existing and proposed and the common outfall trunk, existing and proposed. The City Engineer of Ansonia supplied the following information concerning the design capacity of the proposed treatment plant to be located on the west bank near Division Street: dry weather flow -- 3.5 mgd, and wet weather flow -- 5.2 mgd with approximately 60% originating in east bank sector. The developed areas which are occupied by industrial and commercial establishments and dwelling units have been estimated to be 80 percent of the total. It is estimated that about 25 percent of the dwelling units are multiple family units. No population data has been obtained to estimate the anticipated sewage flows. All design has been based on the figures supplied by the City Engineer.

(2) East bank. On the east bank 635 acres are serviced by the present system. An additional 279 acres will be serviced by the proposed extension, making the total area 914 acres.

(3) West bank. On the west bank the area serviced by the present collector system is 404 acres; that to be serviced by the proposed extension 147 acres, making the total 551 acres.

(4) Both banks. The total area of the existing plus proposed systems on both banks is 1,465 acres.

c. Seepage. The design of seepage flows was based on a seepage factor of 0.08 gpm per foot of dike length per foot of dike height, derived from sub-surface investigations. The land behind the stone masonry walls in the vicinity of the Farrel-Birmingham and the American Brass companies has reportedly been filled with cinders. It may be concluded, therefore, that here the soil is very permeable. During the period of rising river stages in August 1955, the floor of the Farrel-Birmingham Erection Shop was observed by plant personnel to have had many boils which broke through the concrete.

2. DESCRIPTION OF POSSIBLE SOLUTIONS

a. Area 1.

(1) Existing storm drains.

(a) There is in existence a system of storm drains designed to accommodate the runoff from Area 1 and the American Brass Company parking lot. This parking lot was apparently created by filling in part of the old canal. The existing system originates near the Farrel-Birmingham building with catch basins, manholes, and an 8-inch pipe. The flow is directed upstream toward the existing headrace canal used to supply process water to the mill buildings.

(b) The first low spot in the local streets occurs at a bend in Liberty Street, near where the driveway enters the grounds of the American Brass Company. Surface water is intercepted by catch basins, passed through sand traps, and discharged into the pipe under the parking lot. At this point, the pipe under the parking lot increases in size to 36-inch diameter, the capacity of which is estimated to be 15 c.f.s. Under the conditions of the recommended solution the surface runoff to this point will be diverted to a force main, to be discussed under Area 3.

(c) The second low point in the local streets occurs at the intersection of Third and Star Streets. Here again, the runoff is collected in catch basins, passed through sand traps, and discharged into the mainline collector. Here the pipe size remains the same, but by increasing the pitch the capacity is increased to an estimated 40 c.f.s. From the manhole in the parking lot near Third Street the mainline drainage is carried to a manhole in the parking lot near Fourth Street. From this point the mainline drainage turns toward the river and is carried in various sizes and kinds of pipe. Some of these outfall pipes are vitrified clay. The capacity of the outfall system is estimated to be 42 c.f.s. There is indicated on the above referenced plan a proposed connection between the manhole at Fourth Street and the headrace canal; however, no information is available to indicate that this connection was ever made.

(d) A third low spot in the local streets is located at the intersection of Fourth and North Fourth Streets. Surface runoff at this point is intercepted by catch basins, passed through sand traps, and discharged directly into the headrace canal through a 24-inch pipe.

(2) Recommended solution.

(a) In order to keep pumping requirements to a minimum, it is recommended that as much runoff as possible be diverted so as to flow by gravity into the headrace canal. The existing outfall to the river will be abandoned, and the drainage from the parking lot diverted to the process water pipe. The discharge of surface runoff is, therefore, relatively inexpensive because it is to be accomplished using a simple gravity system. Computations for the runoff from this watershed were based on a runoff rate of 0.75 inches per hour per acre.

(b) Normal surface runoff which finds its way to the intersection of Third and Star Streets will be caught by the existing system and carried by a new 36-inch concrete pipe designed to withstand the internal pressure to the headrace canal. Normal surface runoff to the intersection of Fourth and North Fourth Streets will be discharged into the headrace canal by the existing system.

(c) In order to guarantee the interception of surface runoff which may exceed the capacities of the existing storm drains, a concrete curb wall will be constructed, as shown on Plate A-6, along the sideline of Third Street to the edge of the embankment and then along the top of the embankment to the canal. This wall will be carried far enough upstream to permit the water to spill over the embankment without causing erosion of the embankment. The minimum height of the wall above ground will be 12 inches and the profile of the wall will be so established that flow will follow the wall upstream.

(d) The water surface elevation of the standard project flood is 46.0 feet, m.s.l., and the elevation of the crest of the canal embankment nearest the river is in excess of 50.0 feet, m.s.l.

(e) The existing storm drainage system under the parking lot would no longer outfall to the river since the existing outfall line will be removed to the line of protection and the rest plugged and abandoned. The mainline of the system will be connected by a special manhole, which will permit flow in one direction only, to the existing process water pipe originating at the headrace to the mills. Details of this manhole have been left to the final design stage; however, it is expected that an overflow weir arrangement or tide gate can be used to accomplish the purpose. The existing 36-inch pipe will be able to carry the discharge without being relaid to a favorable gradient. It is assumed that the normal water level in the canal will be below the elevation of the parking lot pavement. During periods when the water level in the canal rises higher than the parking lot pavement the 48-inch pipe will be closed by a sluice gate. Process water apparently discharges into one or both of the tailraces described under Area 2. Discharge from these tailraces will be intercepted and conducted to the Maple Street pumping station.

b. Area 2.

(1) Existing drains. The entire area is underlain by an extensive system of storm drains, sanitary sewers, and process water conduits. Some of these pipes, etc., have been constructed by the private companies themselves for their own purposes, and some are municipal. Most of the storm drains, including roof

drains, apparently terminate in the process water tailraces. A few discharge directly into the river. The majority of the storm drains serve the local area, but at least two outfalls to the river are fed surface runoff from the streets above. Nearly all of the sanitary sewers connect to a collector sewer which is linked to the municipal system. A few may discharge into the river, but they are of a minor nature. There are numerous minor process water outfalls and blow-off pipes which discharge directly to the river. No tangible data is now available concerning the size, location, capacity, or use for any of the outfall pipes. All sizing of pipes, etc. for the preliminary design has been based on a rough, office estimate of conditions. An extensive survey must be made to gather sufficient data for design. Runoff quantities were based on the discharge rate of 0.75 inches per hour per acre which will be discharged by gravity during low river stages, and by pumping during high stage.

(2) Seepage. The design of seepage flows is based on a flow rate of 0.08 gpm per foot of head per foot of dike. Except as noted, underdrains will be of 8-inch and 12-inch perforated corrugated metal pipe packed in suitable filters. When flow conditions are such as to cause the 12-inch pipe to reach its maximum capacity, the seepage will be discharged into a collector system and a new line of underdrains will be started. Where the mill buildings rest on the old stone masonry wall along the edge of the river, the space between these walls and the line of protection will be limited. Seepage flows and the flow intercepted from outfall pipes will be carried by pipes placed in this limited space. In order to intercept as much seepage as possible and to reduce the uplift pressure on the mill buildings this line of pipes will be set as low as can be discharged with the flattest, practical gradient. Corrugated perforated metal pipe will be used, up to 12 inches in size, and open jointed, concrete pipe will be used from 12 inches up to 24 inches in size.

(3) Seepage collector and drain interceptor.

(a) The solution to the drainage problem in this area is to intercept all seepage flow, storm runoff, and process water and conduct it to an outfall located near the Maple Street bridge where it will flow by gravity into the river or be pumped during periods of high river stage. Sanitary sewage which now outfalls into the river will be diverted to the sanitary sewer system. Minor outfalls and blow-offs will either be intercepted by the underdrain system and conducted to the collector, or, if this is impractical, they will be gated to prevent backflow during periods of high river stage. The major outfalls will be discussed separately at their proper place in the following discussion. Because of lack of information, it was difficult to determine the points of concentration of surface runoff and the quantities involved. No information was available regarding the discharge of process water, so that pipes were sized according to the best possible estimate of conditions. If there are no large quantities of process water involved, it is believed that the net result will remain the same.

(b) The first major outfall pipe at the upstream end of the system is the 18-inch line from the parking lot of the American Brass Company. As described under Areal, this outfall will be removed or plugged and abandoned.

(c) The second major outfall downstream on the collector is an 18-inch pipe located near station 240. A manhole on the collector will be constructed across this outfall pipe and the flow diverted. The pipe remaining between the manhole and the river will be removed or plugged and abandoned. Flow into this manhole will be from the 18-inch pipe, the 12-inch collector from the first line of underdrains, and the pipe from the second line of underdrains. It has been estimated that an 18-inch pipe leaving this manhole will have sufficient capacity.

(d) The third major outfall pipe is near station 237. This will be a 15-inch pipe and flow in this will be intercepted by a manhole in the collector main located across the outfall line in a convenient position for a change in grade or alignment. No underdrains will enter this manhole. The discharge leaving the manhole has been estimated to require a 21-inch pipe.

(e) The fourth major outfall is a 12-inch pipe located somewhere in the vicinity of station 234. A manhole located over this outfall will serve as a point for alignment change and also as the terminal for the third line of underdrains. The pipe leaving this manhole will be 24-inches in diameter.

(f) The next downstream manhole will be set across an existing tailrace which it will intercept. This manhole will serve as a point for a change in alignment, terminate the fourth and fifth lines of underdrains, and serve as a collection point for the catch basins to be located in the yard between the buildings. The outfall from the tailrace and the 24-inch pipe will be removed or plugged and abandoned. The yard area is a local low point in the terrain, and additional catch basins will be required to service the area after construction of the flood wall. There are two 12-inch outfalls to the river from under the mill building. One is located at about station 226 and the other about station 228. It is anticipated that these outfalls can be intercepted by a pipe set on top of the underdrains and leading to this manhole, or by the underdrains themselves. Discharge from this manhole will require a 36-inch pipe.

(g) Crossing the yard between the mill buildings there is a passageway on a structural steel viaduct. The interceptor-collector drain will be laid under this passageway between the footings, and will then head down the abandoned railroad spur. The crossing under the viaduct will be made at right angles, and two manholes will be provided to make the necessary changes in direction. The 36-inch drain will be laid between the buildings along the abandoned railroad spur into the Farrel-Birmingham property. A manhole will be located at some convenient point to permit a change in alignment. The pipe leaving this manhole will be increased to 48 inches in diameter.

(h) The Farrel-Birmingham plan shows a 14-inch storm drain crossing under buildings 8, 10, and 21, turning upstream under building 22, and outfalling into the river between buildings 23 and 24. This drain seems to service the local streets above the mill. If possible, runoff into this drain will be diverted and included with that in Area 3 as described later. If diversion is impossible, discharge will be intercepted by the main drain where the two pipes cross, and the remainder of the 14-inch line plugged and abandoned. A change in alignment is required in this area so that a manhole will be located at the intersection of the two pipes. The main drain will then follow the main line railroad track to the existing tailrace. Discharge from the old masonry tailrace under the Farrel-Birmingham mill buildings will be intercepted in a special manhole constructed over the tailrace at the intersection of the two lines. Downstream from this manhole the pipe will be increased in size to 60 inches in diameter. Since there are many pipes which empty into this tailrace, it will be necessary to examine each as to source and quantity carried. Storm water which can be diverted to the Area 3 system will be eliminated; sewage will be diverted to the sewerage system; and abandoned pipes will be plugged. The

60-inch pipe will be laid to the gravity outfall and pumping station intake. Some change in alignment will be necessary in order to clear the buildings. This may require that the pipe be laid on a curve, or another manhole installed. Along the riverside wall of the Farrel-Birmingham plant there are several minor outfalls which can be intercepted by the underdrains. Adjacent to the pier footing for the Maple Street bridge there is a new 8-inch corrugated metal pipe which will also be intercepted by the underdrains or diverted directly to the pumping station intake.

(i) In conclusion, a functional system of drainage has been devised based on available information. Since no elevations in the area were firmly established, it was considered impracticable to set invert elevations, etc. As was pointed out above, more tangible discharge information in the final design may alter the design somewhat.

(4) Pumping station.

(a) The Maple Street pumping station will be designed to deliver 50 c.f.s. against a total head of 31 feet. It will have three 18-inch propeller pumps operating at 1160 rpm connected through right angle gear drives to three 80-hp diesel engines. The selected pumps have the following characteristics:

At 580 RPM

Head	Capacity	Horsepower
5.0'	4,800 gpm	8

At 700 RPM

Head	Capacity	Horsepower
5.0"	6,400 gpm	12
10.0	4,900 gpm	15

At 870 RPM

Head	Capacity	Horsepower
5.0'	8,300 gpm	18
10.0	7,400 gpm	24
15.0	6,300 gpm	30
18.0	5,100 gpm	31

At 1160 RPM

Head	Capacity	Horsepower
10.0'	10,900 gpm	44
15.0	10,400 gpm	46
20.0	9,700 gpm	60
25.0	8,800 gpm	68
30.5	7,750 gpm	74

(b) The engine drives will have variable speed governors to permit flexibility of operation, and to reduce as far as possible ponding requirements. The station has been designed so that the pumps can be operated at the rated speed and still have a minimum four minute cycle time. An underground pond is called for in the design of the station, but further analysis of the potential storage in the 60-inch pipe and the tailrace may eliminate the necessity for this storage pond. Each pump will discharge directly into the river through the wall of the station. Protection against backflow from the river will be provided by a flap gate on the end of each discharge line, and a hand operated valve between the pump and the wall of the station. The pump and gate valve will be connected by dresser couplings. The inlet chamber to the station sump will be located across the gravity outfall pipe, and will have a trash rack installed to protect the pumps from debris. Discharge from the inlet chamber into the station sump will be controlled by a suitable sluice gate. The pumping station will be equipped with a 3-ton bridge crane with a hand operated trolley for handling equipment. The 60-inch outfall pipe will be protected against backflow from the river by a tidegate and a sluice gate located in the inlet chamber. The elevation of the invert of the 60-inch outfall pipe has been tentatively set at elevation 15.0 feet m.s.l. The top of the inlet chamber will be covered by an easily removable cover or subway grillage. Riverside protection will be limited to small endwalls or wings to protect the ends of the small and large pipes and their tidegates from floating debris. The river channel will not require special protection because the base of the flood wall will be under the outfall pipes.

(c) From all available information the station should fit the location. Length, up and down the river, should be no problem; width, the distance between the flood wall and the railroad track, may be critical because of the large clearances required by the railroad to prevent fouling. If the available width is insufficient the engines could be located over the gate valves. This arrangement could be made by setting the valves horizontal and using an offset floorstand. The floorstands would be located between the engines, and the rear wall of the station would be moved riverward to provide minimum pump clearance. However, this is not considered the most desirable arrangement.

c. Area 3.

(1) Existing drains.

(a) Most of the area is serviced by a municipal storm drainage system, but no exact information is available because all plans and records were destroyed during the August 1955 flood. Available evidence indicates that drainage which comes down State Street discharges into the Farrel-Birmingham tailrace or is carried through their property to the river. According to the plans, no drainage is carried down Maple Street to Main Street. Surface runoff from the Liberty Street area is discharged into the gravity system located under the parking lot of the American Brass Company. Sanitary sewage is collected in a separate system and conducted to the inverted siphon crossing the river.

(2) Revisions to storm drainage.

(a) The overland flow during times of high rainfall intensity will follow the streets into the pumping stations. It was decided, for reasons of economy, to intercept this flow and use a gravity system to discharge it through a force main directly into the river. This requires that special consideration be given to the method of interception because the interception of overland flow must be definite and complete. It also requires the design of special manholes which can safely withstand pressure from within.

(b) The system will originate with a 54-inch pipe laid under State Street near North Cliff Street. This pipe will intercept all existing pipes and replace them downstream from this point. A series of large-mouthed, gutter-inlet catch basins will be constructed along both edges of the pavement. A continuous catch basin or drop inlet will be constructed across the street from curb to curb, and will have track drains, open bridge decking, or a substantial subway grillage to support traffic. This structure will have sufficient area to intercept all water flowing down the street even though the openings may become partly clogged by debris. The collection structures described will have no sumps, but rather a system of self-scouring pipes discharging into the 54-inch main. Storm drains from intersecting streets and the discharge from existing catch basins on State Street will be diverted to this new 54-inch main drain. This pipe will be laid along State Street to the intersection of North Main Street and from this intersection toward the north along North Main Street with a special manhole at the intersection to make the alignment change. Since State Street is steep, runoff will be flowing with considerable velicity by the time it reaches the intersection, and the sharp turn may cause conversion of much of the velocity head to elevation or pressure head which could lift a standard manhole cover from its frame. The manhole cover and frame should, therefore, be designed with a double cover; one cover to resist the upward force of the water, the other to carry the traffic load. A special frame and cover designed for this purpose will be discussed under the description for Area 6.

(c) The 54-inch main storm drain will be laid along North Main Street to a manhole at the intersection with Liberty Street. The alignment will then change and the drain will follow Liberty Street to the driveway of the American Brass Company. All storm drains and catch basins along North Main and Liberty Street will discharge into the 54-inch storm drain. The bend in Liberty Street near the driveway entrance to the American Brass Company is the low point in the terrain. A system of catch basins and curb-to-curb interceptors will be constructed here as on State Street and all drains, new and existing, will discharge into a manhole.

(d) From the manhole in Liberty Street to the river the pipe will be a 60-inch force main laid under the American Brass Company driveway. Three special manholes with special frames and covers will be installed on this line. One will be located on either side of the railroad track, and the third will be located at the bottom of the driveway. During high river stages, water will stand in all three of these manholes so that the covers will have uplift pressures on them. The force main outfall will have no tidegate and will project only a few inches through the wall and thus will require no protection on the riverside of the flood wall. A heavy steel trash rack which will swing outward and upward, will be provided to prevent debris from the river from being washed into the pipe, and to keep unauthorized persons from invading the pipe. The 54-inch pipe laid under the streets will be reinforced concrete with rubber joints; and the 60-inch force main will be welded steel pipe.

(e) In spite of the length of relatively large pipes involved in the system, the cost of disposing of the surface runoff from the area is about one fourth that of providing a pumping station. A runoff factor of 0.75 inches per hour per acre was used as a basis for flood discharge computations.

d. Area 4.

(1) Existing drains.

(a) The entire area is serviced by a system of municipal storm drains, most of which discharge directly into the river. According to available information, only two catch basins now discharge into Beaver Brook. As in the areas previously discussed, little information is available as to the size, location, and design capacity of the existing storm drains because of the loss of records during the August 1955 flood. Apparently the system was designed for a rate of runoff of 1/2" per acre per hour. The City Engineer made the statement that all new installations or revisions to existing installations will be designed on the basis of a runoff rate of one inch per acre per hour. Several pipes which will be intercepted by the drainage system associated with the local protection project have already been constructed on this basis. Sanitary sewage from the entire area is collected by a separate system, and disposed of by an inverted siphon across the river to the west bank.

(2) Changes to storm drainage.

(a) It is recommended that, as far as possible, changes to the existing storm drainage system be confined to a minimum. The interceptor sewer will be linked to the existing drains so that at times of high river stages the flow will be diverted to the pumping station, but will continue to discharge into the river during the periods of low water.

(b) In order to prevent infiltration and to provide seep rings around the pipes, each outfall will probably require rebuilding between the interceptor and the river. The replacement pipes will be welded steel or cast iron, and each will have a tidegate in the river and a sluice gate on the landside of the protective works. The sluice gate will be located in each overflow manhole at the intersection of each outfall and the collector pipe. In nearly all cases the profile of the collector has been established so that the intercepting pipe is below the outfall pipe. The top of the outfall pipe will be removed in the manhole, and a weir designed and constructed along the side of the pipe so that when the river is high, water will flow over the weir and into the collector pipe below. If, for some reason, the flow in the collector exceeds the capacity of the collector during low water, the reverse process will take place.

(c) There are, in addition to the major storm drain outfalls, several intake or outfall pipes of a minor nature. Many of these pipes, including the only known inlet pipe, have been abandoned and are out of service. These can be removed and other minor pipes which are in service can be intercepted where they cross the collector or the underdrains; the flow will then be completely diverted to the pumping station.

(3) Seepage.

(a) Design was based on the figure of 0.08 gpm per foot of head per foot of dike. Underdrains will be 8-inch and 12-inch perforated corrugated metal pipe packed in suitable filters and set low in the ground to improve their effectiveness. When the flow into the pipe reaches the capacity of the 12-inch pipe, the seepage will be discharged into the collector and a new line of underdrains started.

(4) Seepage collector and drain interceptor.

(a) The system has two trunk collectors; one is upstream of the pumping station, and the other is downstream of the station. The downstream trunk has two branches, one of which services the lower end of the protective works along the river, and the other services the protective works along Beaver brook. Definite quantities of flow from local storm drains which could be expected to enter the interceptor system were unknown, but estimates were made based upon available information. It is assumed that in final design with more information available the elevations and gradients may be changed, but that the same criteria would be used, and the net resulting costs would remain the same.

(b) The upstream end of the collector system will begin at a manhole near station 112 on the east side of the railroad track. This manhole will collect the seepage flow from the first line of underdrains downstream of the Maple Street pumping station. The ground elevation in the vicinity of this manhole is 25 to 26 feet, m.s.l., and the invert elevation in the manhole will be at elevation 18.5 feet, m.s.l. This will permit the underdrain to be set about seven feet below the surface of the ground. The collector pipe leaving this manhole will be 12-inch reinforced concrete.

(c) The first major outfall pipe apparently follows the alignment of Bank Street to the river at about station 109. This outfall will be intercepted in a manhole at the intersection of the two lines. This manhole will be conveniently located to permit a change in line and grade, and to provide a terminal for the second line of underdrains. The invert elevation of the interceptor in the manhole will be 17.6 feet, m.s.l., and the interceptor will be under the outfall pipe, The collector drain leaving this manhole toward the pumping station will be a 27-inch reinforced concrete. The third line of underdrains will terminate in a manhole located at about station 105. The invert elevation in this manhole would be 16.3 feet, m.s.l., and the pipe will continue through the manhole without change in size.

(d) The next major outfall will be a 42-inch reinforced concrete pipe just downstream of the Bridge Street bridge near station 101. A manhole will be constructed at the intersection of the outfall line and the collector drain which will intercept the runoff flow during periods of high river stages. This manhole will collect the seepage from the fourth line of underdrains. The invert elevation of the collector pipe entering from upstream will be at elevation 15.1 feet, m.s.l., and the invert elevation of the 42-inch outfall at approximately 15 feet, m.s.l. The invert elevation of the 48-inch reinforced concrete pipe which will leave this manhole will be at elevation 9.0 feet, m.s.l. The 42-inch pipe will pass through the middle of the manhole with an overflow weir on either side to provide sufficient discharge capacity.

(e) The next manhole in the collector line will be located at about station 99 to provide a convenient change in alignment. A manhole will be located near station 97 to provide a convenient point for an alignment change, and to terminate the fifth line of underdrains. The invert elevation of this manhole will be at 8.3 feet, m.s.l., and a 54-inch reinforced concrete pipe will be used to conduct the flow from the manhole.

(f) The next manhole in line, located near station 91, will be a terminal for the sixth line of underdrains to provide a change in alignment. The 54-inch reinforced concrete pipe will be continued through this manhole, and the invert elevation in the manhole will be 7.7 feet, m.s.l.

Two manholes will be provided on either side of the railroad spur track near station 88 to provide the alignment changes necessary to make a crossing with a reasonable skew angle. The size of the collector will remain unchanged in passing through these manholes, and the collector will have a straight line profile.

(g) The next manhole in line will be located near station 88, and will be the terminal for the seventh line of underdrains. It will be located so as to intercept the flow from a 36-inch outfall pipe shown on the plans of the American Brass Company. No information is now available concerning the use of this 36-inch pipe. If the pipe is used for storm runoff only, regardless of ownership, the flow will be intercepted. Between the manhole at station 88 and station 83 there are several outfalls. These may be active or inactive storm sewers or overflows from the sanitary inverted siphon. If the 15-inch outfall shown near station 87 is an active storm sewer, flow will be diverted to the manhole at station 88. Sewage overflows will be plugged and abandoned. Any other active storm drains will be diverted to the manhole at station 83. The new inverted siphon for sanitary sewage will cross over the storm drainage collector near station 86.

(h) The manhole at station 83 will be so located that it will terminate the eighth and ninth lines of underdrains. At this manhole the collector line will turn inland from the line of protection toward Canal Street. The pipe size leaving this manhole will be 54-inch. Opposite station 83 there will be a manhole located in Canal Street so that the alignment of the 54-inch collector drain can be changed to follow Canal Street to the pond and pumping station.

(i) Another manhole will be required in the vicinity of station 79 and located in Canal Street so that another turn in the collector line can be made to carry the pipe under the railroad spur track into the pond. The gradient of the 54-inch pipe from the manhole at station 97 to the ponding area will be 0.00106 feet per foot. The entire collector drain will be at a low elevation and a flat gradient so that it will clear all existing pipes by passing under them and to permit the underdrain installations to be set low in the ground to improve their effectiveness.

(j) The tenth and eleventh lines of underdrains will be terminated near the driveway into the United Shoe Machinery property. Seepage will be conducted along this driveway to Canal Street and then up Canal Street to the ponding area and pumping station.

(k) The collector drain for the Beaver Brook protective works will originate at a manhole located near station 159. This manhole will be the terminus of the first line of underdrains which is unusually long because of the steep gradient available. A 12-inch reinforced concrete pipe will carry the flow from this manhole. The invert elevation of the manhole will be 17.3 feet, m.s.l. The second manhole in the collector line will be located at a convenient site in Powe Street to permit a change in alignment. The 12-inch reinforced concrete pipe will follow Powe Street to the intersection with Front Street. The third manhole will be located at the intersection of Powe Street and Front Street where it will intercept the flow from the catch basins serving the local area. It has been assumed, on the basis of estimated quantities of seepage and runoff, that an 18-inch reinforced concrete pipe will be sufficient to conduct the flow from this manhole. The invert elevation of the 18-inch pipe at this manhole will be at elevation 16.4 feet, m.s.l. The 18-inch collector drain will be laid under Front Street to a manhole located at the bend in the street near station 156 where the alignment will change.

(l) From the manhole at station 156 the 18-inch reinforced concrete pipe will continue to follow Front Street to a manhole located in the street between stations 154 and 155 of the protective works. This manhole will collect the seepage from the second line of underdrains. The 18-inch pipe will be continued from this manhole following Front Street, and have an invert elevation of 14.9 feet, m.s.l., at the manhole. The next manhole will be located at the intersection of Main Street and Front Street, and will have an invert elevation of 13.4 feet, m.s.l. In addition to the flow into this manhole from the 18-inch pipe along Front Street, there will be flow from the surface runoff from the Main Street area north of Front Street plus the seepage from the protective works and the surface runoff from the Main Street area south of Front Street.

(m) The fourth line of underdrains will be laid at an adverse grade beginning near the Naugatuck River and ending in a manhole on Main Street near station 150. The invert elevation of this manhole will be at 14.6 feet, m.s.l. A 12-inch reinforced concrete pipe will carry the discharge from this manhole. The third line of underdrains will be discharged into a manhole located in Main Street near station 151. The invert elevation of this manhole will be elevation 14.1 feet, m.s.l. The combined seepage flow from the third and fourth lines of underdrains plus the surface runoff from the local area will be conducted by a 15-inch reinforced concrete pipe up Main Street to the manhole located at the intersection.

(n) It was assumed that a 24-inch reinforced concrete pipe will have sufficient capacity to carry the combined flows entering the manhole at the intersection of Main and Front Streets to the manhole which will be located at the intersection of Front and Canal Streets. The invert elevation of this last structure will be at elevation 12.1 feet, m.s.l., and the pipe along Front Street will pass over the top of the 60-inch outfall from the pumping station. At the manhole in the intersection of Canal and Front Streets the flow in the pipe along Front Street will be joined by the seepage and surface runoff from the area below the pumping station, in the corner between the Naugatuck River protective works and the Beaver Brook protective works. From the manhole at the intersection of Front and Canal Streets, the 24-inch pipe will be laid toward the north along Canal Street to a manhole in Canal Street above the pumping station. From the last mentioned manhole, the 24-inch pipe will be laid under the railroad track and discharge into the ponding area. A runoff factor of 0.75 inches per hour per acre was used as a basis for flood discharge computation.

(5) Front Street pumping station.

(a) The station will have three 30-inch propeller pumps operating at 730 rpm and designed to deliver a combined total of 127 c.f.s. at a total head of 30 feet. Each pump will be driven by a 200-hp diesel engine with appropriate right angle gear connections. The selected pumps have the following characteristics:

At 730 RPM

Head	Capacity	Horsepower
5.0	27,800 gpm	90
10.0	26,700 gpm	110
15.0	25,400 gpm	125
20.0	23,600 gpm	145
25.0	21,600 gpm	155
30.0	19,000 gpm	170
31.0	18,000 gpm	175

(b) The engine drives will be constant speed because sufficient pondage will be available to provide a minimum cycle time of approximately six minutes. A dresser coupling and a motor-operated gate valve will connect each pump to the outlet chamber. The invert elevation of the Beaver Brook channel where the outfall from the pumping station enters the channel will be 4.0 feet, m.s.l., and the invert of the outfall at this point will be 5.0 feet, m.s.l. The invert of the gravity flow conduit through the station and also the entrance to the sump will be at elevation 6.0 feet, m.s.l. The outfall pipe will be 60-inch reinforced concrete pipe pressurized by installing concrete rings at each joint.

(c) The outfall pipe will be protected against backflow from the river by a tidegate set into a rectangular recess in the wall so that the gate can swing outward into the full open position without encroaching on the main flow in the brook channel. It is believed that if the gate were to open out into the main channel stream, floating debris could damage the gate and render it inoperative. A 60"x72" pressure unseating sluice gate will be located across the upstream end of the gravity flow conduit and will be used when the pumps are in operation and in the event the tidegate fails to function. A trash rack will be placed in the gravity inlet to the station to protect the tidegate from being jammed by debris floating down the outfall pipe.

(d) The pump sump will have a trash rack across the inlet opening to protect the pumps from debris and each pump will have a screen on it. The inlet opening will be kept closed when the pumps are not actually in use, by a 60"x72" pressure seating sluice gate.

The station will be equipped with a 3-ton bridge crane with a hand operated trolley for moving equipment. In the final design it may be desirable to reduce the ponding requirements by using variable speed governors on the engines.

e. Area 5.

(1) Existing drains. There is an existing storm drainage system on River Street. Available information indicates that the system originates on River Street, near Maple Street, and the flow is carried upstream along River Street to an unknown point of discharge. There is in existence a catch basin and a manhole at the upstream end of River Street. No plans or other definite information is available for this system.

(2) Changes to storm drainage. It is recommended that all overland flow from outside the protected area be intercepted and diverted. The lower Maple Street area is scheduled for redevelopment as part of the Broad Street Renewal Project. If this redevelopment takes place, the profile of Maple Street will be altered in order to carry the overland surface flow past the River Street intersection, and thus away from the protected area. If the Renewal Project does not materialize, the profile of River Street near the intersection with Maple Street will require alteration. A low hump will be created in the street which would prevent flow into the protected area, and will divert it to other discharge locations.

(3) The design of the underdrains was based on the seepage flow rate of 0.08 gpm per foot of head per foot of dike. Underdrains will be 8-inch, 12-inch, 15-inch and 18-inch perforated, corrugated metal pipe packed in suitable filters of granular material. There will be two lines of underdrains; one under the toe of the dike, and the other under toe of the flood wall along River Street. Each line of underdrains will discharge into the pumping station inlet chamber.

(4) Disposal of storm drainage and sewage.

(a) The existing storm sewer will be revised so that the discharge from behind the protective works will be conducted to the pumping station inlet chamber. The remainder of the system which may be outside the protective works will be removed and plugged. Surface runoff which is not intercepted by the existing system will be collected at the roof of the inlet chamber where it will drop into the structure for disposal. Because the area is very flat and given to puddling, it will be advisable to install structures with limited openings to drain the puddles which will form in the surface drain system. A runoff factor of 0.75 inches per hour per acre was used as a basis for flood discharge computations. Any sanitary sewer outfalls to the river will be diverted to the sluice gate compartment of the inlet chamber where the sewage will be carried by a shaped invert to the gravity outfall pipe.

(5) Pumping station and outfall.

(a) The River Street pumping station will have three 8" x 6" sewage pumps designed to give a combined discharge of 6,000 gpm at a total head of 27 feet. Each pump will be powered by a 20 hp electric motor operating at 1150 rpm. The pump operation will be completely automatic. Each pump will discharge into a common 24-inch header pipe with a check valve between the pump and the header. After it passes through the wall of the structure, the 24-inch pipe will be laid up the landside slope of the dike, over the top of the dike, and down the riverside face into a manhole. At the top of the dike there will be an air vent which will exhaust air while the pipe is filling to prevent air bind, and which will admit air to the pipe and break the siphon action. With this air vent the pumps will be operating under conditions of nearly constant head.

(b) Gravity flow during periods of low river stage will be carried by a 24-inch pipe under the dike to the same manhole at which the pipe from the pumps is terminated. From the manhole previously mentioned, a 24-inch pipe will carry the gravity flow or the pump discharge under River Street to the river. This outfall into the river will be protected by a standard straight endwall and will have an invert elevation of 15.0 feet, m.s.l. The invert elevation of the gravity system will be 18.0 feet, m.s.l., at the manhole and 22.0 feet, m.s.l. at the pumping station. Backwater from the river will be excluded from the gravity pipe by a tidegate in the manhole and a 24-inch sluice gate in the inlet chamber of the pumping station. Because of economic considerations and space limitations the entire station will be underground. The structure will be designed to support highway loadings, and it could, therefore, be located in a driveway or service area to the buildings. Access to the pump room will be gained by manholes set in the roof, and movable concrete slabs or heavy steel covers will be placed over each pump so that the equipment can be replaced. No crane or hoist will be installed, but a portable hoist or truck crane can be used for maintenance.

(c) In order to admit surface runoff, a grillage or subway grating will be installed in the roof of the inlet chamber. Estimates of costs were based on the use of 24-inch flanged cast iron pipe, but welded steel pipe with dresser couplings may be less expensive.

f. Area 6.

(1) Existing drains.

(a) In times of intensive rainfall, the municipal storm system reaches its capacity and the overland flow follows the streets to the intersection of Franklin, Jackson and Maple Streets. At this intersection there are now several inlet structures which intercept at least part of this flow. These structures all discharge into a concrete chute which carries the flow down the steep hillside into a brook below. The brook runs along the toe of the slope for about one-half mile, discharging into the river upstream of the American Brass Company parking lot. This brook originates at the rear of the Ansonia Manufacturing Company parking lot, and is fed by several streams from the hillside above.

(b) Overland flow from this area and from adjacent areas will collect in a low spot on Maple Street near the intersection with River Street. The evidence indicates that the flow then follows River Street to the river. Sanitary sewer plans indicate that except for the flood plain behind the proposed protective works, the entire vicinity is service by a municipal system. It has been assumed, therefore, that sanitary sewage from the Ansonia Manufacturing Company will be discharged directly into the river at some point close to the plant.

(2) Changes to storm drainage. Overland flow intercepted at the Jackson-Franklin intersection will be discharged by a gravity force main through the line of protection into the existing brook. Sanitary sewers and storm drains which discharge into the river

from the protected area will be intercepted and the flow diverted to the River Street pumping station where it will flow by gravity through the line of protection or be pumped during high river stages.

(3) Force main.

(a) This system for the disposal of surface runoff was selected for use in order to avoid the necessity of increasing pumping capacity, laying pipe along the side of a precariously steep hill with no room for machinery, and to avoid changing the existing municipal storm drainage system. It was assumed that the existing storm drainage system was designed to accommodate a runoff rate of 1/2" per hour per acre of the 40-acre watershed and, therefore, the proposed force main is designed on the basis of 1/4" per hour per acre runoff. The system will operate by gravity at all times, but the pipes will have considerable head on them when the river is up.

(b) The existing chute from Franklin Street will be removed and replaced with a standard type manhole constructed around the pipes under the sidewalk or as close to the sidewalk as is practical. From this manhole a 24-inch pipe will be laid down the hill to a new manhole constructed at the toe of the slope. Because of its light weight, flexibility, and ease of handling, this pipe will be 24-inch asphalt coated corrugated metal. Special care will be given to the joints to prevent leakage from the inside. From this manhole a 24-inch reinforced concrete pipe pressurized with concrete collars will be laid to the brook at a point outside the dike. A change in alignment at the electric sub-station requires the installation of an additional manhole in the line. All of the concrete pipe and part of the ACCM pipe will be under pressure from within during high river stages. Rather than dig the concrete pipe into the ground and lay it on a flat gradient, thereby creating potential silt pockets, the pipe will be set high and earth mounded over it to protect it from the weather. The outfall to the brook will be protected by a standard type of straight endwall.

(c) The manholes at the toe of the slope will be special reinforced concrete structures designed to withstand the bursting pressure caused by a rise in river level to the top of the dike plus any impact forces due to velocity changes in the manhole. They should also have sufficient structural strength to hold down a special frame and cover which will be anchored to the structure. A double cover, sealed manhole frame and cover will be used for this purpose. The same type manhole frame and covers will be used on the force main in Area 3. The inner cover and the frame flanges will be designed for a head of 30 feet.

g. Area 7.

(1) Existing drainage systems.

(a) Most of the area is serviced by a municipal drainage system. However, the area between the railroad track and the river, except in the immediate vicinity of Division Street, has no drainage system. Percolation and overland flow dispose of surface runoff here. It was assumed that the existing drainage systems were designed to accommodate runoff from a rate of 1/4" to 1/2" per acre per hour. A large part of the contributing watershed is in Derby, Connecticut.

(b) Available information indicates that the contributing watershed was originally serviced by three main drainage systems prior to the construction of Route #8, when these systems were altered. The first of these systems originated on Seymour Avenue, followed a southerly course to Water Street, and discharged into the Housatonic River. The expressway was constructed in a cut past Seymour Avenue which interrupted the normal flow path. The Seymour Avenue drain was altered so that drainage was diverted along the expressway by a 24-inch pipe and discharged on the south side of the embankment at the transition from cut to fill. From this point the discharge is carried by an open brook to the confluence of the Naugatuck and Housatonic Rivers. This brook is also part of the second system to be described.

(c) The second main drainage system was a brook located at the foot of the hill which followed approximately the present alignment of the Ansonia Connector and the expressway to the discharge point at the confluence of the Naugatuck and Housatonic Rivers. During the highway construction the brook was carried under the expressway embankment at the interchange by a 36-inch ACCM pipe. The entire brook upstream of the expressway is now inclosed in pipe. One leg of the brook comes down the hill from the vicinity of Atwater Avenue which also carries some sanitary sewage. The source of this sewage is unknown.

(d) The third primary drainage system follows the alignment of Division Street. It has been revised recently by the installation of 65" x 40" metal pipe arch. This new pipe arch collects the discharge from the drainage system north of the intersection of the Ansonia Connector and Division Street. The pipe arch along Division Street outfalls into the Naugatuck River just downstream from the Division Street bridge. The Route #8 expressway has been constructed in a long cut from just below the Seymour Avenue bridge to the northern end of the city of Ansonia. This cut now forms an intercepting ditch for a substantial watershed having steep hills and rocky terrain. When the installed drainage system reaches capacity the runoff follows the pavement and discharges over the embankment at the lower end of the cut section. This flow has cut its own watercourses on both sides of the expressway embankment to the brook which has been previously described.

(e) Field reconnaissance has determined that substantial and frequent concentrations of overland flow occur at the intersection of Clifton Avenue and Cook Street. At this point there is a shallow depression on Clifton Avenue, and Cook Street has a steep down gradient from this low point, so that water follows the street to the end house a few hundred feet from Clifton Avenue. Flow then goes around the house and down the hill to the Ansonia Connector. Apparently, the new Division Street pipe arch is supposed to intercept this flow and conduct it to the river directly.

(2) Changes to storm drains.

(a) It is recommended that, as far as possible, changes to the storm drains be kept to a minimum. It will be necessary, however, to divert the Division Street pipe arch and the 36-inch pipe which parallels the Ansonia Connector. These lines will be intercepted and the flow conducted to the pumping station by ditch. This statement presumes that all sanitary sewage will have been removed from the system.

(3) Seepage.

(a) Design of the underdrains and the computation of seepage flows was based upon the figure of 0.08 gpm per foot of head per foot of dike. The underdrains will be 8-inch and 12-inch corrugated metal pipe packed in suitable filters. When the seepage flows have reached the capacity of the 12-inch pipe, the seepage will be discharged into the collector pipe, and a new line of underdrains will be started.

(4) Collector and interceptor.

(a) The quantities to be intercepted from storm drains is unknown, and no attempt was made to estimate them because of the lack of data. For several reasons, which will be discussed under the heading of sanitary sewers, the existing trunk sewer to the outfall will be rebuilt and relocated. This reconstruction makes it possible to coordinate the profiles and alignments of the sanitary sewer and the collector drain.

(b) The first line of underdrains will begin on the hillside near the Ansonia Connector; it will cross over the top of the existing sanitary sewer; then under the railroad embankment; and terminate at the first manhole of the collector line, station 88 on the river centerline. Where this underdrain crosses the sanitary sewer the invert elevation will be 18.0 feet, m.s.l. The invert elevation of the first manhole will be 15.6 feet, m.s.l., and a 12-inch plain concrete pipe will conduct the flow to the next manhole in line. The second manhole in the collector line will be located near station 84 where it will collect the flow from the second line of underdrains. Also feeding into this manhole will be the surface runoff from the local area which will be collected by a catch basin or drop inlet located at the low point in the terrain. The invert elevation of this manhole will be 14.0 feet, m.s.l. A 24-inch reinforced concrete pipe will presumably have sufficient capacity to carry the discharge from the manhole.

(c) The 24-inch sanitary sewer siphon will cross the collector drain between the second and third manholes. The top of the sewer pipe will be at about elevation 9.0 feet, m.s.l., and the invert of the collector pipe will have an approximate elevation of 13.0 feet, m.s.l. These elevations depend on the exact point of the crossing, but there will be sufficient room between the two pipes. The third manhole in the collector line will be at station 80 on the river centerline. It will serve as the terminal for the third line of underdrains, and to insure that the collector pipe will cross the sewer main at a right angle. The invert elevation of the third manhole will be 12.4 feet, m.s.l; the invert elevation of the fourth manhole, also at station 80 but on the opposite side of the sewer, will be at elevation 12.2 feet, m.s.l. The top of the sanitary sewer where the two pipes cross is at about elevation 10.0 feet, m.s.l., and the invert of the collector will be at 12.3 feet, m.s.l. This two-foot separation between the two pipes was considered the desirable minimum. The pipe leaving the third and fourth manholes will be 27-inch reinforced concrete. The fifth manhole in line will be located at about station 76 on the river centerline and will collect the discharge from the fourth line of underdrains. The pipe size will remain 27-inch at this manhole, and the invert elevation will be at 10.5 feet, m.s.l. The sixth manhole in the collector line will be at station 71. The seepage from the fifth line of underdrains will be fed into this manhole. The invert elevation of the manhole will be 8.9 feet, m.s.l., and the pipe leaving will be 27-inch reinforced concrete.

(d) A catch basin or drop inlet will be constructed at the low point in the terrain in the vicinity of station 69 to collect the surface runoff from the local area. Surface water collected by this structure will be carried across the top of the sanitary sewer by a 24-inch reinforced concrete pipe to a manhole in the interceptor line at about station 69. This manhole will be situated so that it will provide a good hydraulic entrance for the flow from the inlet structure. The invert of the manhole will be at elevation 8.5 feet, m.s.l. The pipe leaving the manhole will be 36-inch reinforced concrete.

(e) The eighth manhole in line will collect seepage flow from the fifth line of underdrains, and will be located in the vicinity of station 65+50. The invert elevation will be 5.4 feet, m.s.l., and the pipe size will be 36-inch. The ninth manhole in the collector line will be located at about station 61+50. It will receive the seepage flow from the sixth line of underdrains. The invert elevation will be at 5.4 feet, m.s.l., and the pipe leaving the manhole will be 36-inch reinforced concrete. The next manhole, the tenth in the collector line, will be located just upstream of Division Street at about station 57. It will collect the seepage from the seventh line of underdrains. The pipe size will be 36-inch, and the invert will be at elevation 3.8 feet, m.s.l. The tenth and eleventh manholes will be located on either side of the Division Street roadway embankment so that a 36-inch pipe can be jacked under the road between the two. Although reinforced concrete was used for estimating purposes, the pipe actually installed may be welded steel or cast iron. If soil conditions prohibit the jacking technique, a 48-inch diameter tunnel lined with corrugated multi-plate tunnel liner will be driven. In this case a 36-inch reinforced concrete pipe can be laid inside the tunnel, or the tunnel can be lined with concrete inside the multi-plate.

(f) The elevations of the storm and sanitary sewers between the fourth and tenth manholes have been set so that the connecting pipes from each underdrain can be laid over the top of the sanitary sewer to the collector manholes. The discharge from each of these connecting pipes will drop inside the manhole. The underdrains themselves, therefore, will be set at the lowest possible elevation which will permit this arrangement. Downstream of Division Street the discharge will be carried by open ditch, fenced in to avoid accident, with a bottom width of 6 feet and side slopes of 2:1. The ditch will be about nine feet deep. The elevations of the bottom of the ditch will be about 3.5 feet, m.s.l. near Division Street and 2.5 feet, m.s.l. near the pumping station. A straight standard endwall will be located at the head of the ditch to protect the 36-inch outfall pipe. The discharge end of the pipe will be covered by an outward lifting trash rack to keep animals and children from invading the pipe. Although the drainage plan and computations indicate that the underdrains between Division Street and the pumping station will be intercepted by the sanitary sewer, it will be better to conduct the seepage flow across the top of the sanitary sewer to discharge directly into the ditch. This arrangement would not permit the underdrains to be set as deep into the ground, but there should still be enough depth available so that their effectiveness is not lost. The downstream end of the ditch will be connected to the pumping station by a reinforced concrete box culvert. This culvert will carry the access road for trucks and other traffic to the pumping station and the dikes. The inside dimensions of the barrel will be 60" x 96".

(g) Between the pumping station and the end of the protective works the seepage flow from the underdrains will be discharged directly into the pumping station storage pond or into the intercepting ditch. The 36-inch storm drain which now parallels the Ansonia Connector will be intercepted and the flow diverted to the pumping station by ditch. The pipes under the Expressway embankment outside the dike can be plugged and abandoned; the pipe under the dike will be removed. A 36-inch or 42-inch pipe culvert can be constructed under the Ansonia Connector to a ditch between the Connector and the railroad track. This ditch will also collect seepage flow and local surface runoff. It has been estimated that a 42-inch pipe culvert will carry the flow under the railroad to the pond. The easiest procedure for installing this culvert under the railroad is by jacking steel pipe. The bottom of the storage pond will be sloped toward the pumping station to insure complete drainage, and a small ditch will be cut into this sloping bottom from the culvert under the railroad to the pumping station. The above description assumes that all sanitary sewage will have been eliminated from the 36-inch drain.

(h) In order to prevent substantial quantities of overland surface flow from entering the protected area, thus requiring an increase in pump capacity, the dike will be relocated away from the expressway embankment. It should be possible to turn the dike to the northwest to terminate in high ground. This will exclude from the protected area the water spilling over the embankment from the expressway and from the brook at the southerly end of Atwater Avenue.

(i) If it is impractical to relocate the end of the dike at this location, a gravity force main would be installed from an intercepting structure near the end of Atwater Avenue and the flow discharged outside the protective works. In addition, a granite or cast-in-place barrier curb should be constructed along the edge of the expressway shoulder from the Seymour Avenue bridge to a point well downhill from the end of the dike. There the flow can be chuted down the side of the fill from the end of the barrier curb. The barrier curb will be the standard type used on highways. A runoff factor of 0.75 inches per hour per acre was used in the flood discharge computations.

(5) Pumping station.

(a) The Division Street pumping station will contain three 36-inch propeller pumps operating at 580 rpm. Each pump will be powered by a 4-cylinder diesel engine rated to deliver 320 horsepower at 1200 rpm. Appropriate right angle gear drives will connect the engines to the pumps. The station has been designed so that the pumps will deliver a combined discharge of 170 c.f.s. against a total head of 30 feet. The pumps and impellers selected have the following characteristics when operating at the speed of 580 rpm which was selected for the design.

At 580 RPM

Head	Capacity	Horsepower
5.0'	40,000 gpm	120
10.0'	38,400 gpm	150
15.0'	36,200 gpm	170
20.0'	33,200 gpm	195
25.0'	29,600 gpm	215
30.0'	25,400 gpm	230

(b) The gravity outfall and pressure conduit will be a 60" x 96" rectangular reinforced concrete conduit with concrete joint rings. It will have sufficient structural strength to support the truck traffic which will pass over it. The invert of the barrel at the outfall in the river will be at elevation 2.0 feet, m.s.l., and at the pumping station will be 2.5 feet, m.s.l. Both of these elevations are below tide level. The outfall at the river will have a 60" x 96" double hinged tidegate with the end of the culvert protected by an endwall design and shaped to protect the tidegate from being damaged by debris. The gravity outfall conduit will be closed by a 96" x 60" sluice gate when the pumps are in operation, or in the event that the tide gate should fail to function. The station layout is such that water entering the 60" x 96" conduit from the ditch will be passed through the station naturally and discharged into the river. In the event of high water the sluice gate at the gravity entrance to the station will be closed. The water will then flow out of the gravity conduit into the pond and back through the trash racks into the station sump from which it will be pumped.

(c) The pumping station will be a wet sump design, and in order to keep the sump dry during periods when the pumps are not in use, a 96" x 60" pressure seating sluice gate will be installed to close the sump inlet. Each pump will have a screen over the inlet, a dresser coupling, and a motor operated gate valve between the pump and the wall of the conduit. The station will have a 3-ton bridge crane with hand-operated trolley.

(d) There will be ample space and volume available for ponding to permit a long cycle of operation. The bottom of the pond will be pitched toward the station to prevent puddles, and the lowest elevation at the station will be 4.5 feet, m.s.l. The maximum level of water will be 10.0 feet, m.s.l., with a desirable maximum of 8.0 feet, m.s.l.

3. DESCRIPTION OF SANITARY SEWERAGE

a. Description of sewerage conditions.

(1) Existing sewerage system.

(a) Nearly all of the populated area in the community is serviced by a municipal sanitary sewer which is divided into two principal districts, the east bank and the west bank. The discharge from both of these districts is combined and conducted to a common outfall on the west bank. The present outfall is a 24-inch cast iron pipe located near the site of the proposed Division Street pumping station. The combined flow is carried from the junction of the two systems, located near station 80, to the outfall by a 24-inch brick conduit with an estimated capacity of 4 c.f.s. The main trunk on the west bank is a 27-inch pipe fed by main branches on Broad Street, Lester Street, and Clifton Avenue. Of these branches the first two combine their flow in a 24-inch pipe which was relocated by the Connecticut State Highway Department in conjunction with work on the Ansonia Connector. Parts of this sewer as relocated are outside of the protective works and will be under water during flood.

(b) Sewage from the east bank is carried across the river by an inverted siphon. The condition and capacity of the siphon are unknown. Two primary trunks feed to the siphon: one follows Canal Street and the railroad to Maple, North State, and North Main Streets. The other follows the general alignment of Central Street with branches along Main, Powe, Beaver, and Jewett Streets. Another sewer system presently discharges into the river from the east bank near the Division Street bridge. Plans call for this sewer to be carried across the river, probably by siphon, into the outfall trunk.

(2) Proposed municipal changes to sewerage.

(a) According to the available information, it is planned to expand the existing system to service other areas in the community. Much of the expansion will be incorporated into the existing system. A new inverted siphon crossing is proposed downstream of the Division Street bridge. Existing plans indicate that the present 24-inch cast iron outfall near the open air theater will be abandoned, and the trunk line carried farther downstream to an outfall near the confluence of the Naugatuck and Housatonic Rivers. Most recent information indicates that there is to be a sewage treatment plant constructed behind the protective works just upstream of Division Street. Land for the purpose has already been acquired.

(3) Recommended changes to sewerage system.

(a) In Area 1 the existing sewers discharge their flow directly into the inverted siphon and have no influence on the proposed protective works; it is anticipated that no changes will be required.

(b) In Area 2 the sewerage system which serves the industrial buildings is linked to the municipal system. There may be some sewers, minor in nature, which discharge directly into the river. If this is the case and if it is possible to do so, these will be diverted into the municipal system.

(c) In Area 3 the gravity force main and the alterations to the storm drains have no bearing on the sanitary system; no changes in the sewerage are anticipated.

(d) In Area 4 there are no major changes to the existing system which can be anticipated except that the inverted siphon will require reconstruction. It may be necessary to rebuild the 8-inch sewer which crosses Beaver Brook near Cheever Street Extension.

(e) In Area 7 the majority of the system will remain as is. It will be necessary to reconstruct or alter the main 24-inch brick outfall trunk, the inverted siphon and its junction, and the 24-inch trunk sewer near Clifton Street.

b. Description of alterations to sewerage systems.

(1) Clifton Avenue-Broad Street sewer.

(a) The 24-inch trunk sewer from the Clifton Avenue-Broad Street area, which was relocated by the Connecticut State Highway Department as it parallels the Ansonia Connector, will be altered to prevent excessive infiltration during high river stages. This sewer from the intersection of the Ansonia Connector and

Clifton Avenue lies between the Connector and the railroad track. In this area the elevation of the top of dike will be 37.0 feet, m.s.l. and there are two sewer manholes and several hundred feet of 24-inch pipe, probably vitrified clay, which are below this elevation. The rim of one manhole is now at 24.1 feet, m.s.l., and the rim of the other is at 35.3 feet, m.s.l. The 24-inch pipe continues downstream under the proposed dike to join the 27-inch trunk headed toward the river. Excessive seepage or a breach in the system during high water could jeopardize the protected area.

(b) The recommended solution is to provide a means for closing off the flow under the dike and to construct an emergency outfall to the river. A new manhole will be constructed across the line adjacent to the riverside toe of the dike and as high as the dike. There would be a catwalk from the top of the dike to the top of the manhole. A 24-inch bronze mounted or stainless steel mounted gate valve will be installed in the manhole so that the pipe under the dike can be closed to flow. In order to relieve the sewage in case of emergency, another pipe with a valve will be set in the manhole several feet above the normal flow and extended to the water's edge. Normally the gate valve on the emergency outfall will be closed, and that on the main line left open; in an emergency, the status of the two valves will be reversed. In order to reduce infiltration the frames and covers of the existing manholes will be removed and replaced. The pipe under the dike will have seep rings installed.

(c) At the new manhole the invert elevation of the main line will be about 14.5 feet, m.s.l., and the invert of the emergency outfall will be at 17.0 feet, m.s.l. At the river the emergency outfall will have an invert elevation of 16.0 feet, m.s.l., and a 24-inch tidegate.

(2) Inverted siphon.

(a) Because the river channel will be relocated and widened, the existing sanitary sewer siphon must be relocated and reconstructed. It will originate in a new manhole to be constructed across the brick sewer on the east bank in the vicinity of the existing manhole. The terminus on the west bank will be in a new manhole constructed in the vicinity of station 83 across the existing 27-inch trunk sewer coming from the Ansonia Connector. The pipe will be 24-inch bell and spigot or mechanical joint cast iron pipe embedded in concrete. It will be set below the improved river bottom. The capacity will be about 6 c.f.s., with a minimum velocity of 2 f.p.s., and a minimum hydraulic gradient of 0.0008 feet per foot. There will be 24-inch gate valves installed in the manholes on either end of the pipe, so that if the siphon should be breached between the dikes during high water, the inflow can be stopped. There will be sufficient difference in the level of the water surfaces on both ends of the pipe to guarantee scour during periods of low flow.

(b) The correct sequence of operations will permit new construction to proceed without interrupting the existing service. The manhole on the west bank will be constructed around the existing 27-inch pipe which will be left intact and supported until the new outfall trunk and the new inverted siphon have been completed. In like manner, the manhole on the east bank will be constructed around the brick sewer which will not be opened until all new construction is finished. On the east bank the outfall pipe and the new inverted siphon will have inverts several feet below that of the 27-inch line from the Ansonia Connector.

(3) Outfall trunk sewer.

(a) The existing 24-inch brick sewer will be abandoned because of the following reasons:

1. The security of the existing brick conduit, especially against bursting pressure, is doubtful. A breach in the pipes subject to flooding by the river may, at least temporarily, cause heads in the sewer of 20 feet or more. The sewer may also be frequently subjected to considerable surcharge.

2. A considerable length of sewer must be rebuilt in order to get it away from the dike.

3. A second inverted siphon would be required to cross under the proposed storm drain. This would cause additional head losses.

4. With the new sewer the friction losses would be less than they are now with the brick sewer.

5. The sewer must cross the storm drain, and by making the crossing as far upstream as possible it is easier to get a greater difference in elevation between the two, making for a better crossing.

6. The City of Ansonia will be required to rebuild the system in order to satisfy the growing demands for capacity, and to bring the sewer line into their proposed treatment plant.

(b) The relocated sewer will be designed as a gravity force main with special frames and covers on the manholes. The pipe will be 24-inch reinforced concrete with single rubber ring-gasket joints. Since concrete will be used, those industrial plants which discharge strong acids or other wastes harmful to concrete should be required to treat their effluent. The force main will begin in the manhole at station 82 where the inverted siphon and the 27-inch trunk from the Ansonia Connector join. The invert of the 27-inch pipe in this manhole is about elevation 9 feet, m.s.l., and the invert elevation of the 24-inch pipe leaving the manhole will be at 7.08 feet, m.s.l.

(c) The next manhole downstream in the force main will be located as near as practical to the landside toe of the dike at about station 77. This manhole will be constructed to effect a change in alignment. The invert elevation will be at elevation 6.36 feet, m.s.l. The next manhole required for an alignment change will be at station 69, and the invert elevation will be 5.37 feet, m.s.l. The next manhole in the line will be located at station 47 where the sewer will take a new heading into the pumping station. The invert elevation here will be 3.78 feet, m.s.l. The last manhole in the line will be located at station 47 where the sewer will take a new heading into the pumping station. The invert elevation here will be 2.17 feet, m.s.l. The pitch of the sewer invert from the pumping station upstream to the junction manhole will be at the rate of 0.0015 feet per foot. Because the outfall from the station is in the tidal water the invert of the pipe at the entrance to the station and at the outfall will be set at 2.0 feet, m.s.l.

(4) Sewage pumping station.

(a) The sanitary sewer pumping station has been designed as an addition to the Division Street storm drainage pumping station, but it could be established as a separate unit elsewhere. In the final design it may be located adjacent to the proposed treatment plant. The station was designed so that, with the maximum total head and with one pump inoperative, the remaining capacity would be 75% of the estimated maximum requirements. Since the main trunk sewer can be used as a force main with a potential head to at least elevation 15.0 feet, m.s.l., the total head against which one pump must operate is greatly reduced and the potential standby capacity then exceeds 100%. In addition, the use of the main trunk sewer as a force main permits less use of the pumps during periods of moderate increase in river stage.

(b) The station will have two dry pit, vertical, angle flow 16-inch pumps. The pump operating speed will be 860 rpm maximum. Each pump will be powered by a 50 hp electric motor designed to operate at 900 rpm synchronous speed. A 40 kw standby generator will be provided. For flexibility of operation the electric motors will be designed for variable speed operation. The pumps and impellers selected for use have the following characteristics:

At 585 RPM

Head	Capacity	Horsepower
5.0'	3,700 gpm	15
10.0'	2,500 gpm	15
15.0'	0 gpm	15

At 695 RPM

Head	Capacity	Horsepower
10.0'	4,900 gpm	20
15.0'	2,300 gpm	20
20.0'	100 gpm	20

At 850 RPM

Head	Capacity	Horsepower
15.0'	6,300 gpm	40
20.0'	4,800 gpm	40
25.0'	2,800 gpm	40
30.0'	740 gpm	40
32.0'	100 gpm	40

The maximum size of solids which the pumps can handle is six inches.

(c) During periods of high river stage when pumping is required, the sewage enters the inlet chamber to the pumps through a 30" x 48" sluice gate which will normally be kept closed keeping the chamber dry. Each pump will discharge through a check valve into a common header conduit which will also be kept dry during low river stages by a 30" x 48" sluice gate. A small sump pump will be provided and used to empty each of these chambers after the pumping operations have been completed. Back-pressure from the river will be controlled by a third 30" x 48" sluice gate which will be installed across the gravity flow concuit. In order to provide adequate ventilation, wells will be constructed over the conduit on each end of the building. These wells will extend through the roof of the building and will be watertight. The sluice gates and their stems will be located in these wells, and the motor-operated floor stands will be set over the wells on th e roof of the building.

(d) Conduits within the station and the outfall conduit will be reinforced concrete with collars at the joints. Conduit area will be increased to 48" x 48" inside dimensions to facilitate construction and to reduce losses. The sewer outfall conduit will end in the same endwall structure used for the storm drain. A 48" x 48" tidegate will be installed at this endwall. Movement of equipment in the station will be accomplished by a 2-ton gantry crane with a 2-ton trolley hoist. With this apparatus, machinery can be moved into the bay where the larger pumps will be set, and then the overhead bridge crane can be used for further transport.

4. ALTERNATES CONSIDERED

a. Area 1. A brief comparison of costs precluded completely the use of pumps to dispose of runoff. Some consideration was given to the advisability of replacing the system of pipes now outfalling to the river from the American Brass Company parking lot with one large pipe. This alternate was abandoned because of the required length and the difficulty to be encountered in placing. Moreover, replacing the pipe still does not solve the whole problem. In nearly all of the areas studied a runoff rate of 1" per acre per hour was first selected because the City of Ansonia is using that figure in all new installations. The use of this figure for the drainage behind the protective works was considered to be economically inadvisable.

b. Area 2. In this area first considerations located the pumping station at the foot of the American Brass Company driveway adjacent to their river bridge. Because of the invert elevation of the Farrel-Birmingham tailrace and because of the general slope of the watershed, the site was abandoned in favor of Maple Street. In Areas 2 and 4 consideration was first given to the advisability of pumping all of the runoff, but the stations required became so costly that the gravity force system was devised, and Area 3 was created.

c. Area 3. In the solution to the problem for Area 3, two other outfall locations were considered for the force main. Consideration was given to the vicinity of the Maple Street bridge as a point of discharge. Limited space eliminated the location from consideration. Another site considered was at Railroad Avenue. Here the added length of pipe plus the extra cost associated with laying large pipe in the streets of a business district eliminated this location from final consideration. In both of the alternates described some provision would have to be made to drain Liberty Street. Also in Area 3 the collector pipe was first laid out so that it originated at the low spot on Pleasant Street near State Street. This is actually where it belongs and if the B/C ratio is not too seriously affected this alternate should become the recommended solution.

d. Area 4. In Area 4 consideration was given to the elimination of the Canal Street pumping station and combining facilities on the west bank near the open air theater. Drainage would have been conducted by pipe to the Beaver Brook culvert as originally laid out. The Beaver Brook culvert originally went between the buildings of the United Shoe factory. The drainage would then go by inverted siphon under the Beaver Brook culvert and across the river, the siphon being two 48-inch steel pipes embedded in a concrete cradle. On the west bank the siphon would discharge into a ditch or into a twin box culvert. The plan of combining facilities was abandoned because the decreased cost of pumping did not offset the cost of the siphon.

e. Area 5. In Area 5 first consideration was given to the use of a wet pit pumping station with axial flow pumps, but the plan was abandoned because of the possibility of sewage in the system, the high cost of a structure to house axial flow pumps, the relatively high operating heads, the lack of space for an overhead structure to house the pumps, and the availability of an inexpensive standardized commercial pumping unit. Consideration was also given in Area 5 to the use of cast iron pipe for the gravity flow discharge and for the pressure pipe over the top of the dike. The estimate is based on this material because there were no firm prices available for steel pipe. It is believed that final analysis will indicate that steel pipe with dresser couplings is less expensive.

f. Area 6. There were no alternates considered feasible for this area.

g. Area 7.

(1) In Area 7 consideration was given to the advisability of utilizing the low land adjacent to the river as a pond to reduce the pumping requirements. It was estimated that sufficient pondage should be available to store 12 inches of runoff from the watershed. The idea was abandoned because the storage requirements were so great that they destroyed the enhancement value of the protective works.

(2) Two pumping station sites were considered. One consideration involved the use of a single station with collectors feeding water from upstream and downstream of the station. The station site would be just upstream of Division Street adjacent to the proposed sewage treatment plant. It was found that adverse gradients were too severe and the plan was abandoned. Another plan involved the use of two stations; one located just above Division Street, and the other situated in the rear of the open air theater. Each station would have disposed of the runoff from the areas upstream of the station. The upstream station was to also serve the sewage requirements. This plan was abandoned because of the economics. It was found that the cost of two stations exceeded the cost of the increased pipe size. A second reason for abandonment of this plan was that no reasonable system could be devised to operate in the interim period between the construction of the protective works and the construction of the sewage treatment plant. It was assumed that the treatment plant would not be constructed for some time after the dikes had been built, and the City of Ansonia desires the sewage to be discharged as far downstream as possible.

(3) In conjunction with the recommended design in Area 7, consideration was given to the use of a box culvert to transport the storm drainage from Division Street to the pumping station in order not to take too much land from the theater. This idea was abandoned in favor of the open ditch because the cost of the culvert exceeded the value of the property lost.

APPENDIX B

FLOOD LOSSES AND BENEFITS

APPENDIX B

FLOOD LOSSES AND BENEFITS

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FLOOD LOSSES AND BENEFITS

1. DAMAGE SURVEYS

A detailed damage survey was made in the main flood areas of the lower Naugatuck River Basin immediately after the flood of August 1955. In view of unprecedented flood stages and damages associated with this flood, losses were referenced to the 1955 stages. By an office review, the results of this survey were correlated with the data obtained after the 1938 and 1949 floods. Additional surveys were conducted during 1958-59 to obtain more detailed flood damage information in the project areas and to obtain information for enhancement and higher utilization studies.

The damage survey consisted essentially of door-to-door interviews and inspections of the several hundred residential, commercial, industrial and other properties affected by flooding. The recorded information included extent of the areas flooded, description of properties, nature and amount of damages, depth of flooding, high-water references and relationships to prior flood stages. Damage estimates were generally furnished by property owners or tenants. Investigators used their own judgment in modifying these estimates and also made estimates where owner or tenant estimates were not available. Sampling methods were used where properties of the same general type were subject to the same depth of flooding.

Sufficient data was obtained to derive losses for (1) the 1955 flood crest, (2) a stage 3 feet higher, (3) the stage at which damage begins, referenced to the 1955 flood crest, and (4) intermediate stages where marked increases in damage occurred.

2. LOSS CLASSIFICATION

Flood loss information was recorded by type of loss and by location. The types of losses included urban (residential, commercial, public), industrial, highway, railroad and utility.

Tangible primary losses were evaluated. Primary losses comprise (1) physical losses, such as damage to structures, machinery and stock, and the cost of cleanup and repairs; and (2) nonphysical losses, such as unrecoverable loss of business and wages, cost of temporary facilities and increased cost of operation. Primary losses resulting from physical damage and a large part of the related nonphysical losses were determined by direct inspection of flooded properties and evaluation by property owners, field investigators, or both. Where nonphysical portions of primary losses could not be determined from available data, estimates were based upon an established relationship between physical and nonphysical losses for similar properties in the area.

No evaluations were made of either secondary or intangible damages. Secondary damages, those incurred outside the immediate flood area under study, include such items as business loss, loss of utilities and transportation facilities, and increased cost of travel and shipment of goods. Intangible losses include such items as loss of life, hazards to public health, and impairment of national security.

3. RECURRING AND PREVENTABLE LOSSES

Stage-damage and discharge-damage relationships were developed to reflect the magnitude of recurring losses at varying stages of flooding above and below the reference flood. The recurring losses used in developing the stage-damage relationship reflect economic and physical changes in the area since the August 1955 flood. The recurring August 1955 losses remaining after reductions by the authorized and recommended reservoirs would be prevented by the recommended local protection project.

4. ANNUAL LOSSES

Estimated recurring losses in the Ansonia area were converted to average annual losses as a basis for determining average annual benefits for use in economic evaluation of the studied projects. These annual loss estimates were derived by correlation of stage-damage, stage-discharge, and discharge-frequency relationships to produce damage-frequency relationships in accordance with standard Corps of Engineers practice.

Table B-1 shows the average annual losses remaining after reductions by the 7-reservoir system for the Naugatuck River Basin and those remaining after addition of the Ansonia protective works. Curves used for computation of average annual losses and benefits in the east bank and Division Street protection areas are shown on Plates B-1 through B-7.

TABLE B-1

RESIDUAL AVERAGE ANNUAL FLOOD LOSSES

ANSONIA AREA
(Jan. 1960 Price Level)

Zone	Average Annual Losses after Reservoir Reductions	Average Annual Losses after Local Protection
East Bank	$197,000	$50,000
West Bank	91,000	32,000
Totals - Both Banks	$288,000	$82,000

5. ANNUAL BENEFITS

Average annual flood damage prevention benefits were derived for the main damage zones in the Ansonia area by determining the difference between average annual losses remaining after reductions by the 7-reservoir system and the average annual losses remaining after protective works. Table B-5, at the end of this appendix, summarizes the annual flood protection and enhancement benefits accruing to the recommended project.

6. ENHANCEMENT BENEFITS

a. General. An analysis of flood losses and benefits in Ansonia revealed that substantial development of currently idle lands and higher utilization of vacant industrial and commercial space can be expected to follow the construction of the flood protection project. The value of this enhancement is an additional benefit to the project. There is a continuing demand for suitable manufacturing space and lands developable for that purpose. In the past year the State of Connecticut has added over 300 new manufacturing plants which have provided 3,600 jobs. The experience of six major floods since 1927, climaxed by the disastrous floods of 1955, have discouraged similar developments in flood-prone areas.

An investigation of past, present, and potential future use of idle lands and vacant industrial and commercial space was made to obtain data for the economic analysis. In the course of this investigation, useful data was obtained from responsible sources which included city officials, bankers, real estate brokers, industrial managers, merchants and the Connecticut State Development Commission. This study revealed a general need for developable land and industrial and commercial space and a specific potential for use of space in the flood area after construction of flood-protection measures.

Manufacturing plays the major role in the economy of the Naugatuck River Basin. This valley comprises the leading nonferrous metal manufacturing area in the nation and produces over one-third of the nation's brass and bronze. Machinery, rubber products, watches, and professional instruments are the other principal products of the region. The area has traditionally been a source of highly skilled labor. Ansonia's industries and commercial functions play an essential role in the economy of the basin.

The growth trend of the State of Connecticut is reflected by the fact that 10.8 percent of manufacturing employment in 1955 was in firms opened in Connecticut since 1947. In the Naugatuck valley, however, only 3.6 percent of such employment in 1955 was in newly established activities.

The State possesses several significant resources which provide a favorable economic climate for such growth and which may be expected to encourage continued expansion. It is well located for serving the large markets on the East Coast and in the Middle West, and for supplying important overseas markets which can be expected to increase. Transportation facilities are adequate and are continually being improved. The labor force is varied and highly skilled. The proximity of the State to New York City and Boston means that it is an advantageous place to locate industries which utilize the specialized skills, including engineering, design, and research, which are available in these areas. The political and social attitudes have been found generally attractive to industry. These favorable statewide conditions are also true of the Naugatuck valley and the city of Ansonia except for two major differences - the fear of floods and the scarcity of land suitable for industrial development. In a study of the economic outlook for the Naugatuck River valley made in 1956 by the Economics Department of Yale University for the Connecticut State Development Commission, it is noted that "the fear of the recurrence of the floods of 1955 in the river valleys is perhaps the greatest cloud on the industrial horizon." It is also noted that, in the Naugatuck valley, "the relative scarcity, in the areas receptive to industry, of land suitable for industrial sites," helps to explain the tendency for the growth to occur in areas other than the valley itself.

The project recommended will encourage utilization of significant land areas and idle space in Ansonia and Derby and thus help fill the existing need.

b. Development of idle land. There are two areas of idle land on the west bank of the river which can be expected to have an important value with the recommended project in effect. There is a parcel of some 30 acres upstream of Division Street in Ansonia and one of about 4 acres downstream of Division Street in Derby. Due to the location of these lands in the flood plain, they are presently in no productive use. These lands are adjacent to established highway and rail transportation and utilities. Process water is available from the river. Labor skilled in the metals industry is available. The lands are in the heart of the existing primary and fabricated metals industries. These factors all indicate that these lands, when protected, would very likely be intensively used for metal fabrication and warehousing. Although a precise determination of the market value of this type land is difficult to make because of the many factors of influence, it is estimated that a capital value of $14,000 per acre represents a reasonable estimate for industrial purposes in this locality. This is substantiated by views expressed by the State Development Commission. The lands have negligible current value.

Protection of these lands would produce an estimated average annual enhancement benefit of $840 per acre based upon an annual net yield of six percent.

On the east bank of the river there are separate parcels of idle land located on the fringe of the central business district which have a total developable area of approximately 110,000 square feet under the recommended plan. These parcels are individually of a size, shape, and location suitable for commercial development. The business district of Ansonia serves the shopping demands of the surrounding towns as well as those of the city of Ansonia. Recognizing this, numerous inquiries have been received from chain store systems, individual entrepreneurs, and bowling and recreational center interests relative to the use and development of these areas. To date, the flood history has discouraged investment and development.

The current market value was estimated on the basis of several recent transfers of generally similar type of land both in and outside of the flood area and of reported sales offerings. A conservative estimate indicates present market values of about $1.25 per square foot, which an increase to about $2.50 per square foot after flood protection. This protection would produce an estimated average annual enhancement benefit of $0.075 per square foot based upon an annual net yield of six percent.

Table B-2 presents the location, area, type of potential development, estimated amount of current and post project market values and average annual enhancement benefits which would be realized after construction of the recommended project.

c. Utilization of idle industrial space. There are two existing buildings with space available for rent within the area to be protected. Both are located on the east bank near the downstream end of the project. The former occupants moved after the 1955 flood and, although manufacturing and warehouse space is in demand, the flood history has precluded reoccupancy of these buildings to date, even at relatively low rental charges. There is a total of 115,000 square feet of floor space. The average annual gross rental for this type of space is about $0.60 per square foot with an estimated 15 percent of this required for services such as heat, water, and maintenance, leaving a net rental income of $0.51 per square foot as the initial net return from rental of this space. Without additional investment or inordinately expensive maintenance, this return would be reduced to zero at the end of the economic life of the protective works. The average annual benefit to the project from utilization of vacant industrial space is estimated at $29,300.

TABLE B-2

ENHANCEMENT OF IDLE LAND

LOCATION	LAND AREA	TYPE OF DEVELOPMENT ANTICIPATED	ESTIMATED MARKET VALUE Current	ESTIMATED MARKET VALUE Post-Project	AVERAGE ANNUAL BENEFITS
West Bank					
Upstream of Division St	30 acres	Industrial	Negligible	$420,000	$25,200
Downstream of Division St	4 "	"	"	56,000	3,400
East Bank					
Main St - between Front and Chestnut Sts	74,000 sq ft	Commercial	$ 92,500	185,000	5,550
Main St - between Central and Cheever Sts	30,000 sq ft	"	37,500	75,000	2,250
Canal St - corner of Water St	6,000 sq ft	"	7,500	15,000	450
Totals			$137,500	$751,000	$36,850
				(rounded)	$36,900

d. Utilization of idle commercial space. The main business district of Ansonia was severely affected by the 1955 floods. Thirteen establishments, vacated after the floods, have not been reoccupied. The history of active retail merchandising in the area indicates that flood protection would result in reoccupancy of the idle stores. The stores have been rehabilitated and some of them improved but, owing to the flood hazard, new enterprise declines to occupy this space.

There are 350 feet of idle commercial space frontage on the main business street. A survey of comparable rental values of this type of retail property indicates that a gross return of $10 per month could be realized for every foot of frontage. The provision of utilities and services are estimated at 15 percent of this, resulting in an initial net return of $8.50 per month for every front foot. Without additional investment, this return is assumed to be reduced to zero at the end of the economic life of the protective works. This average annual benefit is estimated at $17,800.

e. Summary of enhancement benefits. The annual benefits to be realized from development of idle lands and utilization of idle industrial and commercial space are summarized in Table B-3.

TABLE B-3

SUMMARY OF AVERAGE ANNUAL ENHANCEMENT BENEFITS

	East Bank	West Bank	Total
Idle land	$ 8,300	$28,600	$36,900
Industrial space	29,300	-	29,300
Commercial space	17,800	-	17,800
Total	$55,400	$28,600	$84,000

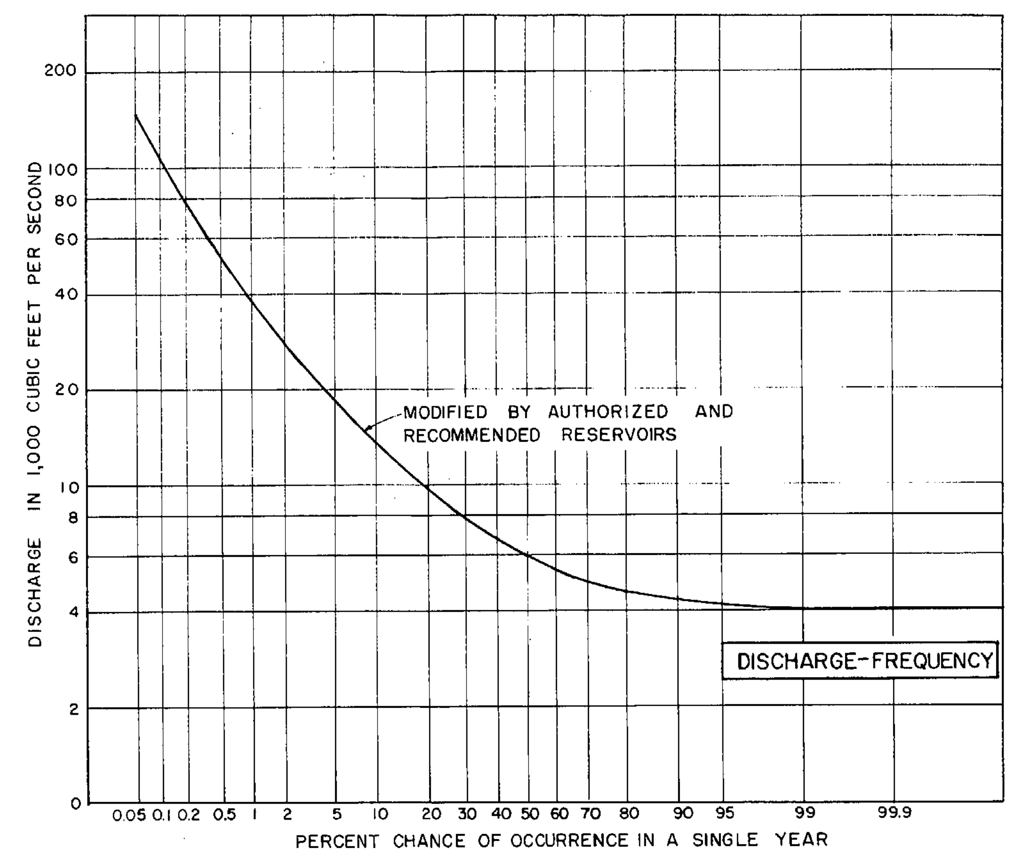
7. SUMMARY OF BENEFITS

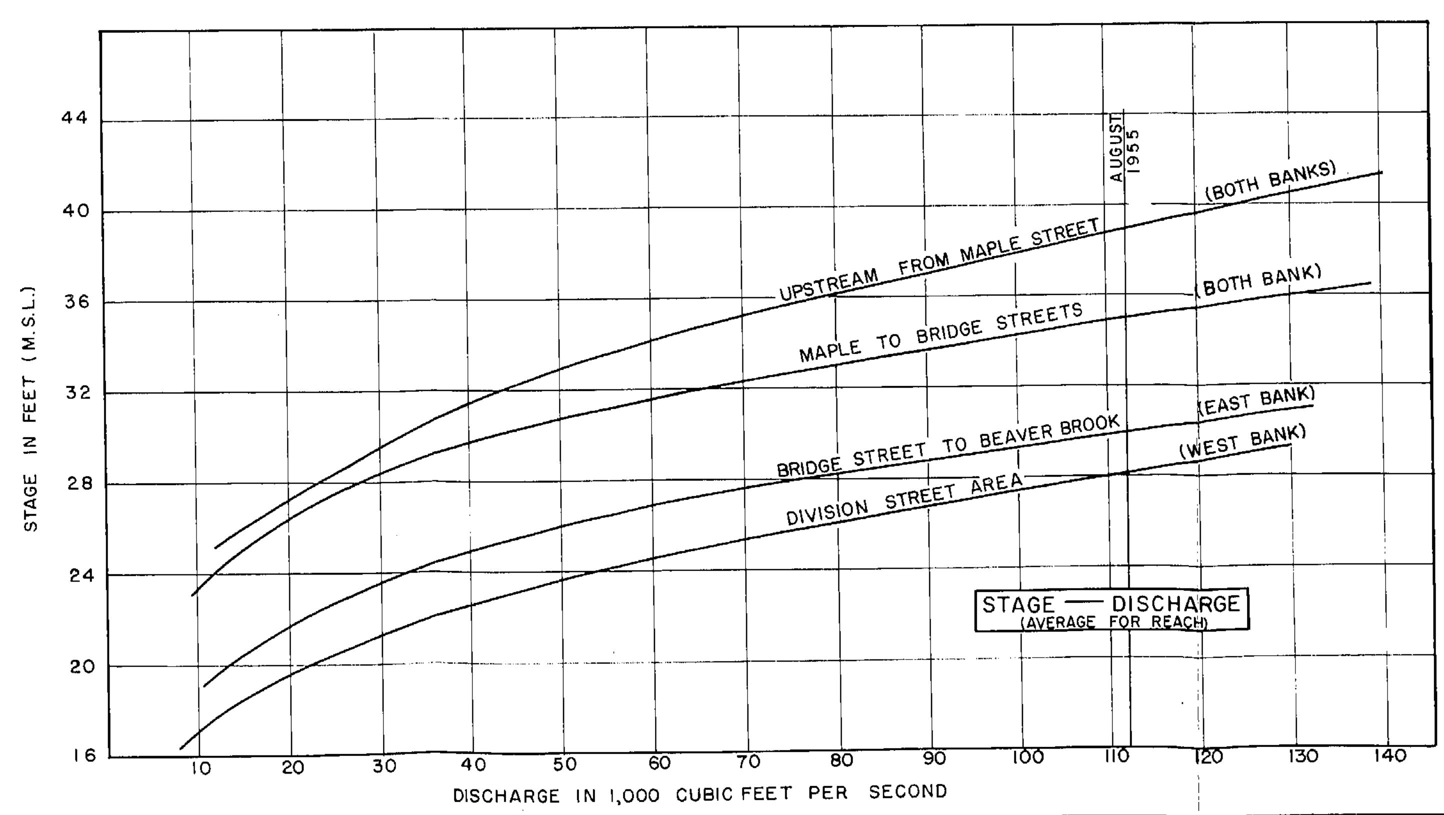
Total annual benefits for the studied Ansonia-Derby local protection projects are presented in Table B-4.

TABLE B-4

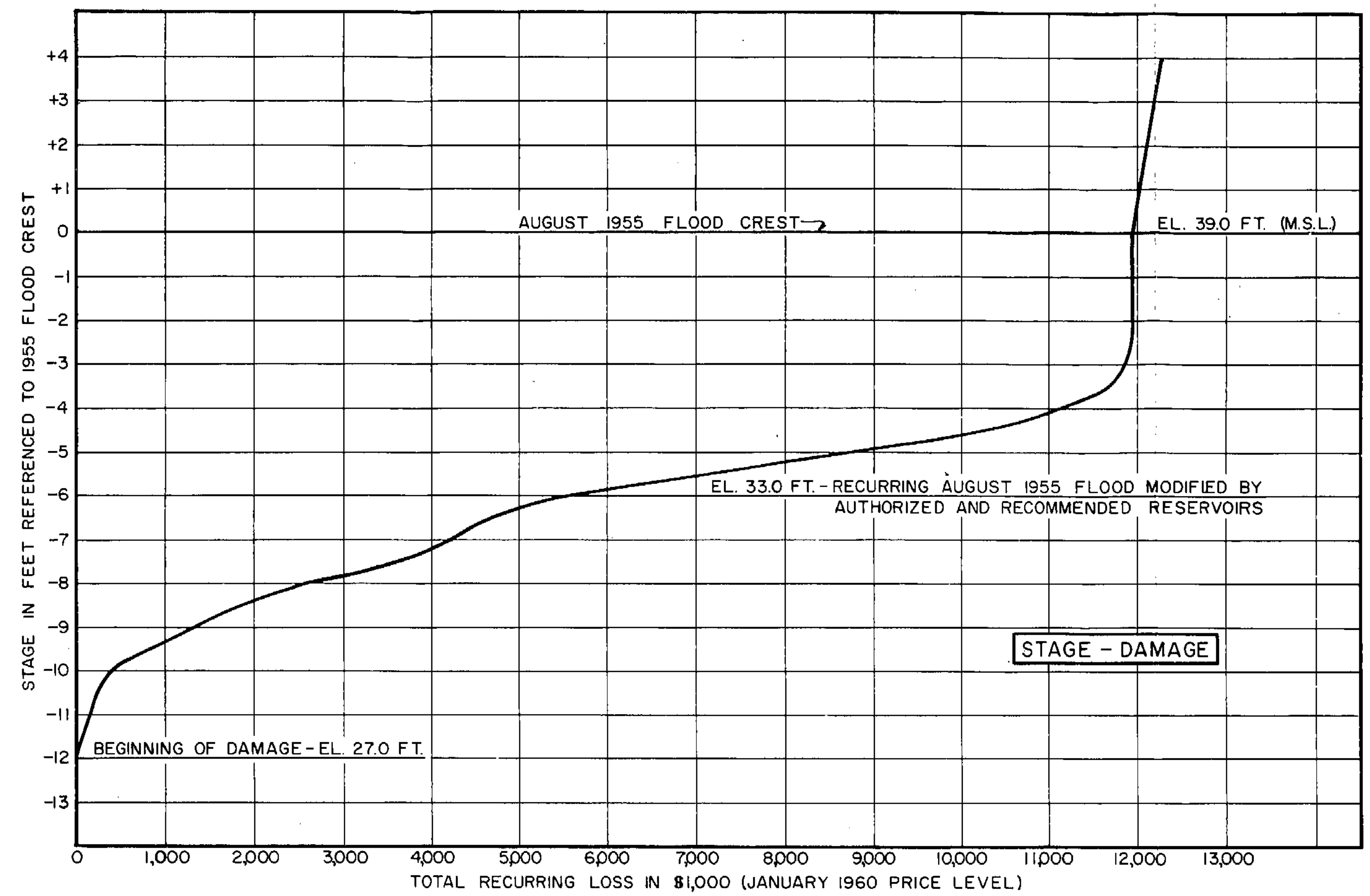
SUMMARY OF AVERAGE ANNUAL BENEFITS

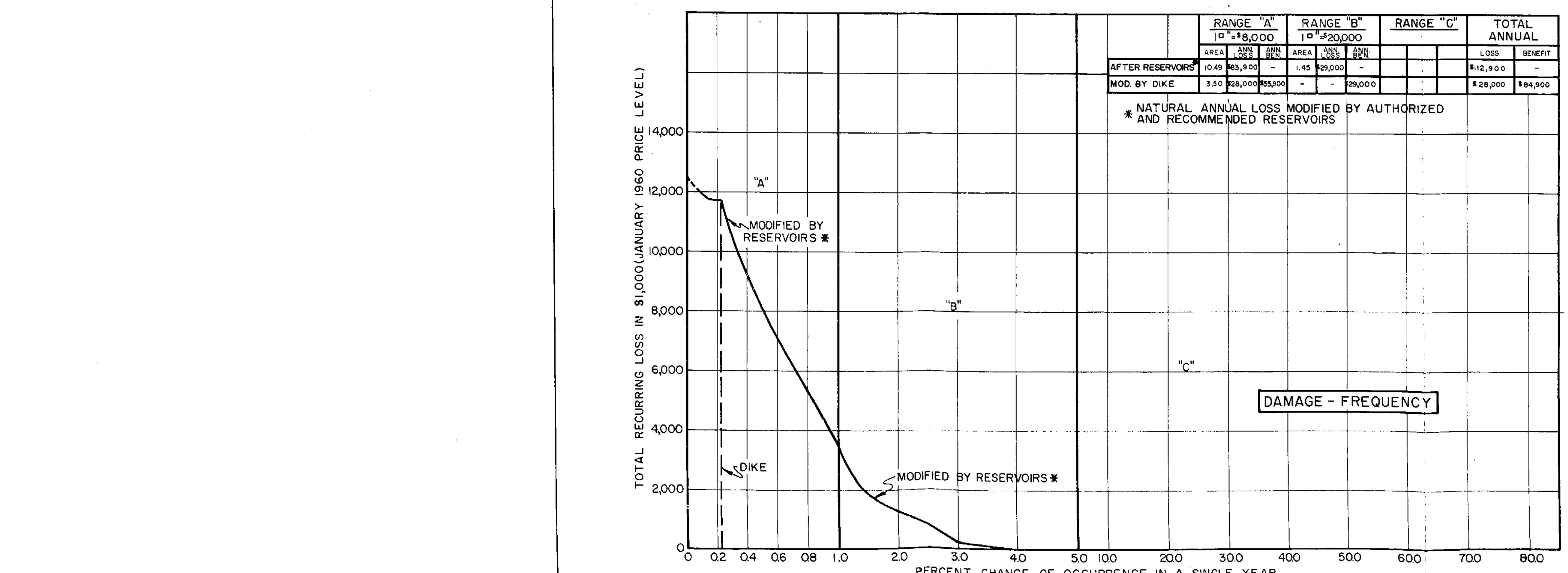
	East Bank	West Bank	Total
Flood damage prevention	$147,000	$59,000	$206,000
Enhancement	55,400	28,600	84,000
Total	$202,400	$87,600	$290,000





HOUSATONIC RIVER FLOOD CONTROL
ANSONIA–DERBY, CONNECTICUT
LOCAL PROTECTION
DISCHARGE-FREQUENCY
AND RATING CURVES
NAUGATUCK RIVER CONNECTICUT



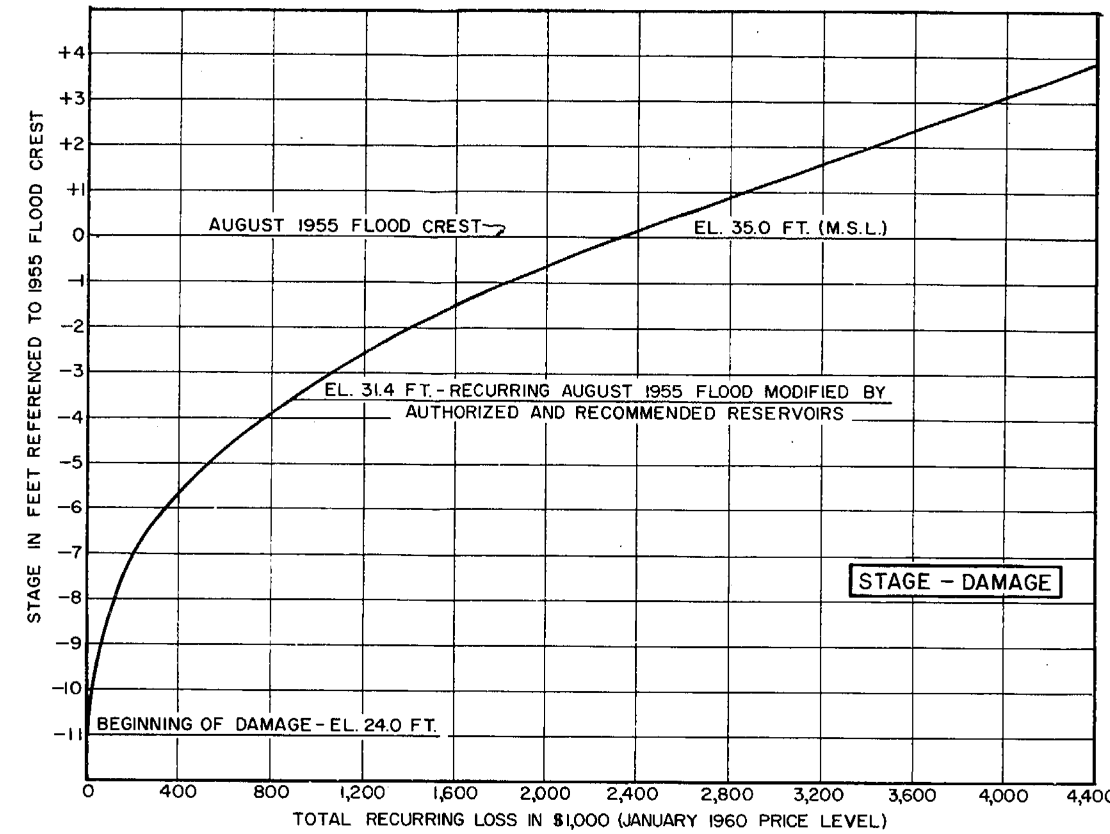


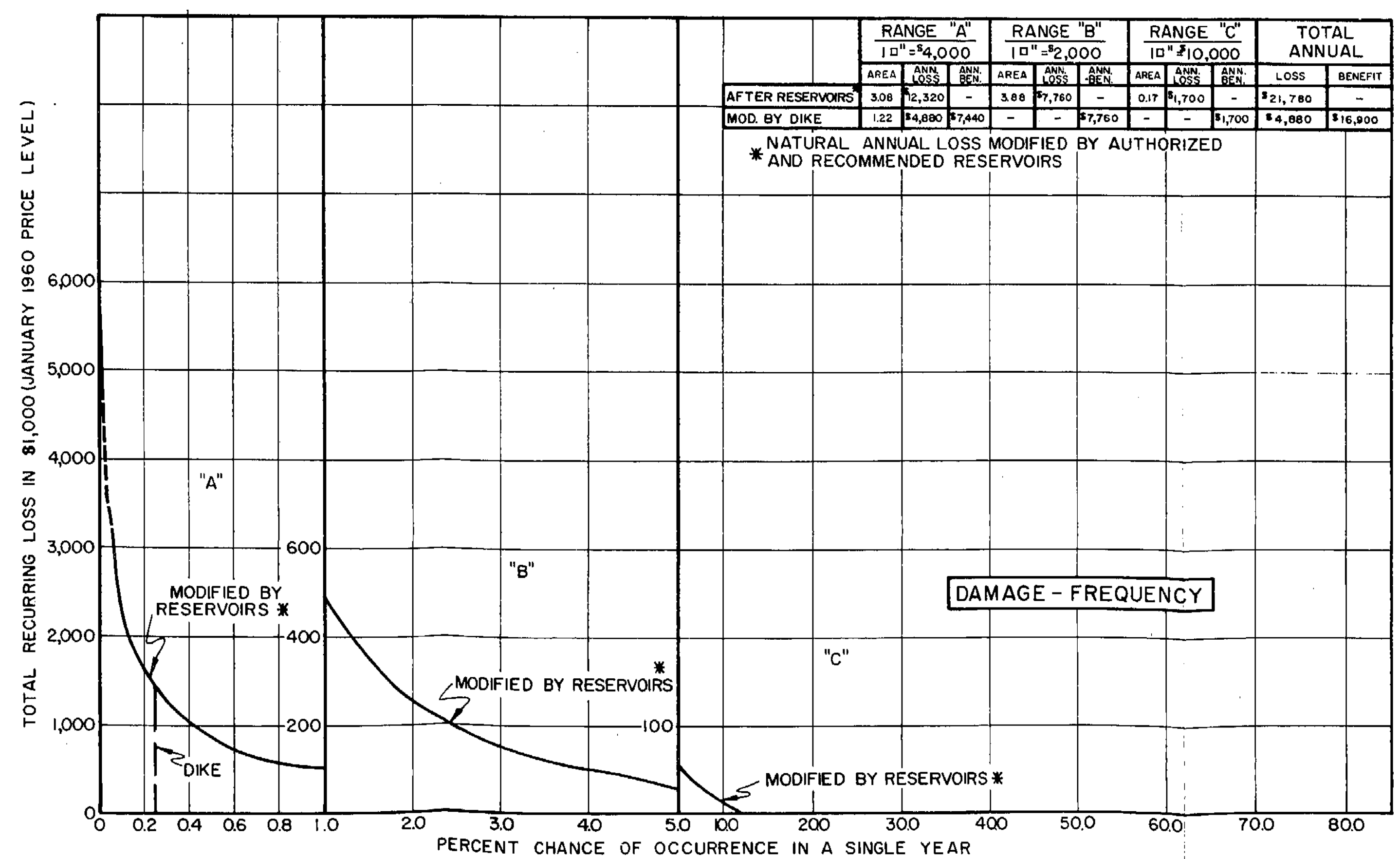
	RANGE "A" 1□"=$8,000			RANGE "B" 1□"=$20,000			RANGE "C"			TOTAL ANNUAL	
	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.				LOSS	BENEFIT
AFTER RESERVOIRS*	10.49	$83,900	-	1.45	$29,000	-				$112,900	-
MOD. BY DIKE	3.50	$28,000	$55,900	-	-	$29,000				$28,000	$84,900

* NATURAL ANNUAL LOSS MODIFIED BY AUTHORIZED AND RECOMMENDED RESERVOIRS

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE - DAMAGE AND
DAMAGE FREQUENCY CURVES
EAST BANK
UPSTREAM FROM MAPLE STREET
NAUGATUCK RIVER CONNECTICUT

PLATE NO. B-2

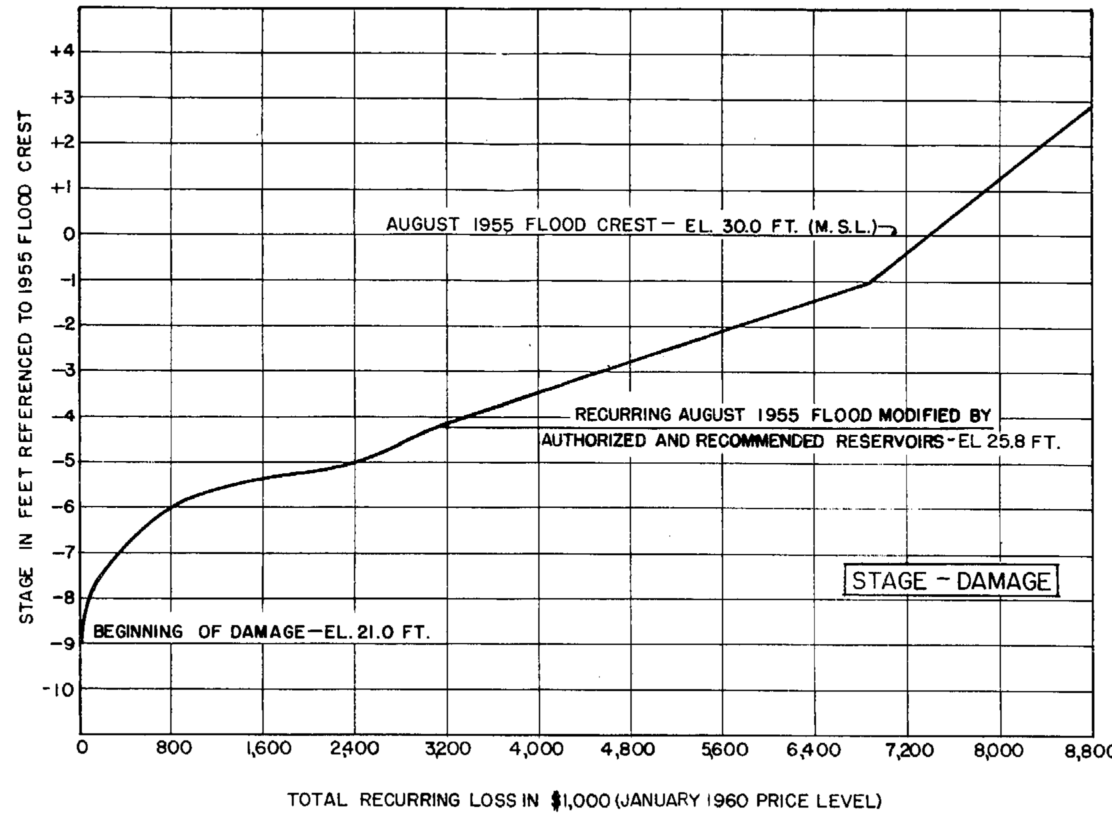


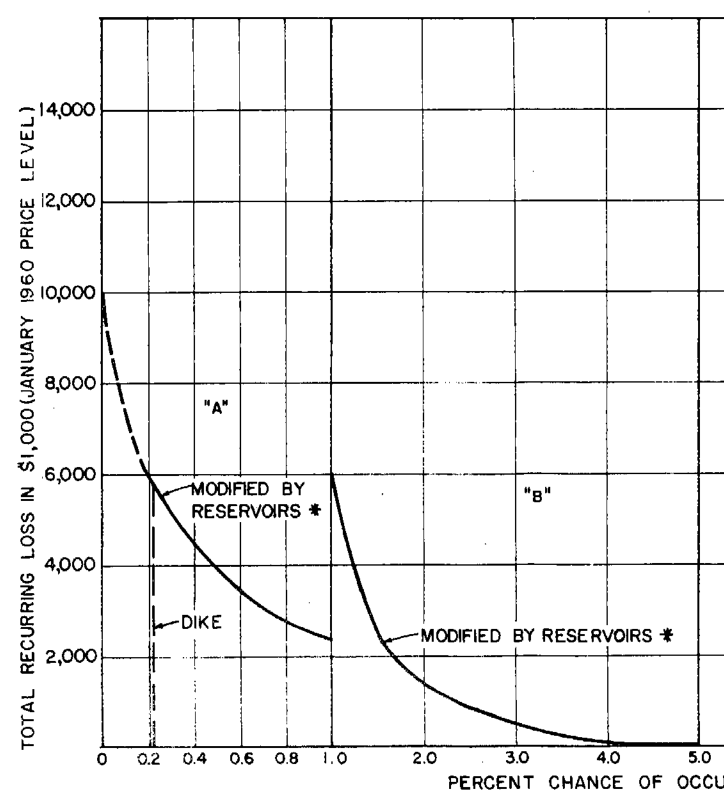


	RANGE "A" 1□" = $4,000			RANGE "B" 1□" = $2,000			RANGE "C" 1□" = $10,000			TOTAL ANNUAL	
	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	LOSS	BENEFIT
AFTER RESERVOIRS*	3.08	*12,320	-	3.88	$7,760	-	0.17	$1,700	-	$21,780	-
MOD. BY DIKE	1.22	$4,880	$7,440	-	-	$7,760	-	-	$1,700	$4,880	$16,900

* NATURAL ANNUAL LOSS MODIFIED BY AUTHORIZED AND RECOMMENDED RESERVOIRS

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE-DAMAGE AND
DAMAGE-FREQUENCY CURVES
EAST BANK
MAPLE STREET TO BRIDGE STREET
NAUGATUCK RIVER CONNECTICUT

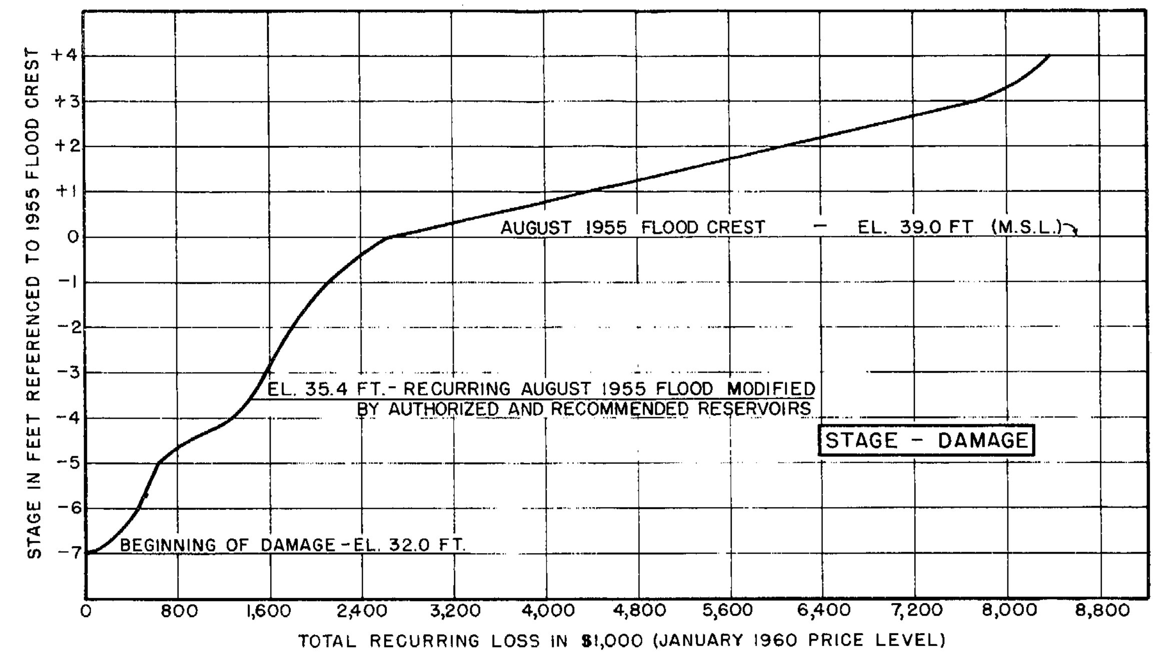


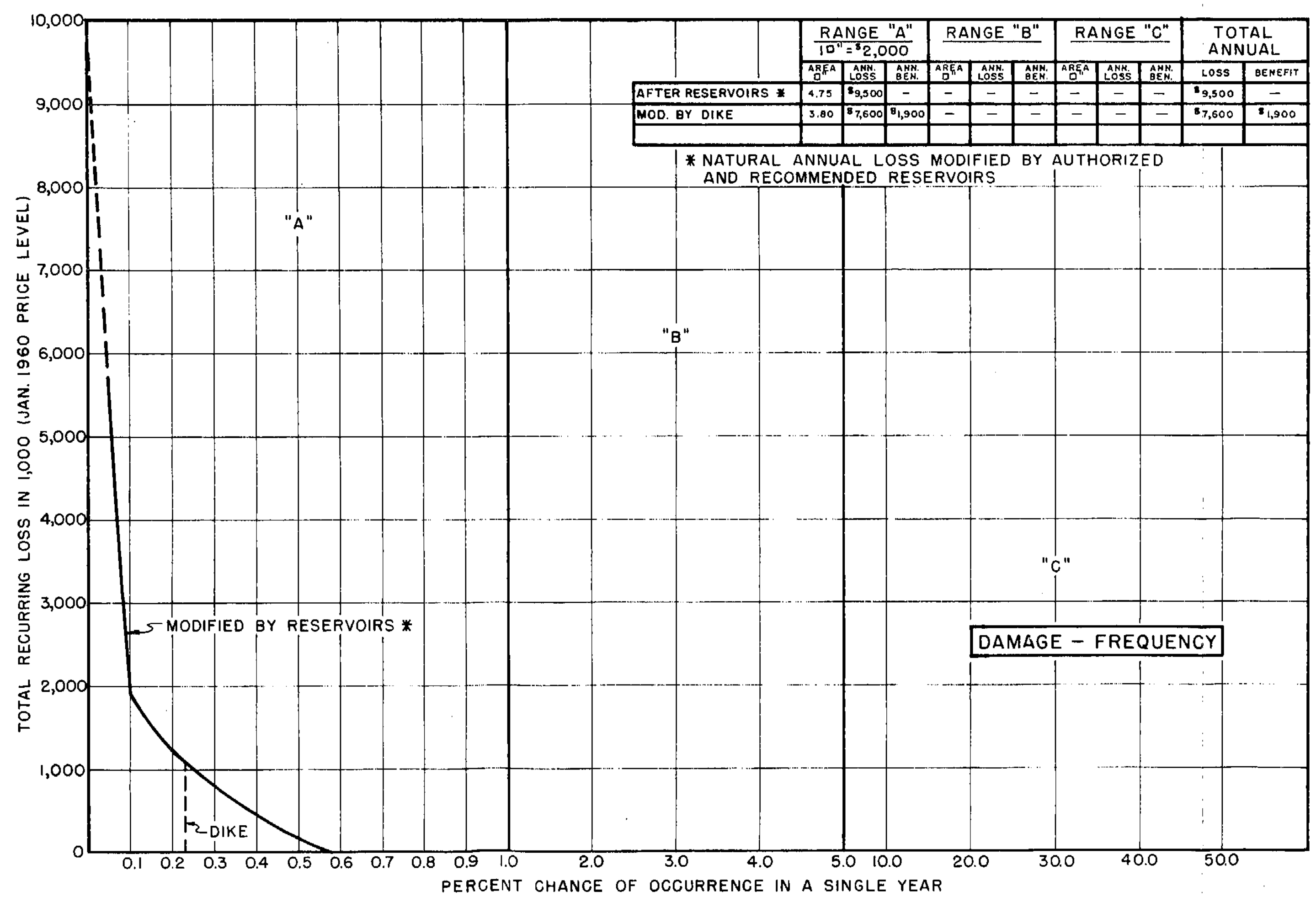


	RANGE "A" 1□" = $8,000			RANGE "B" 1□" = $8,000			RANGE "C" 1□" = $40,000			TOTAL ANNUAL	
	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	LOSS	BENEFIT
AFTER RESERVOIRS*	5.54	$44,320	-	2.16	$17,280	-	0.01	$ 400	—	$62,000	-
MOD. BY DIKE	2.10	$16,800	$27,520	-	-	$17,280	-	-	$ 400	$16,800	$45,200

* NATURAL ANNUAL LOSS MODIFIED BY AUTHORIZED AND RECOMMENDED RESERVOIRS

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA – DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE-DAMAGE AND FREQUENCY-DAMAGE CURVES
EAST BANK
DOWNSTREAM FROM BRIDGE STREET
NAUGATUCK RIVER CONNECTICUT

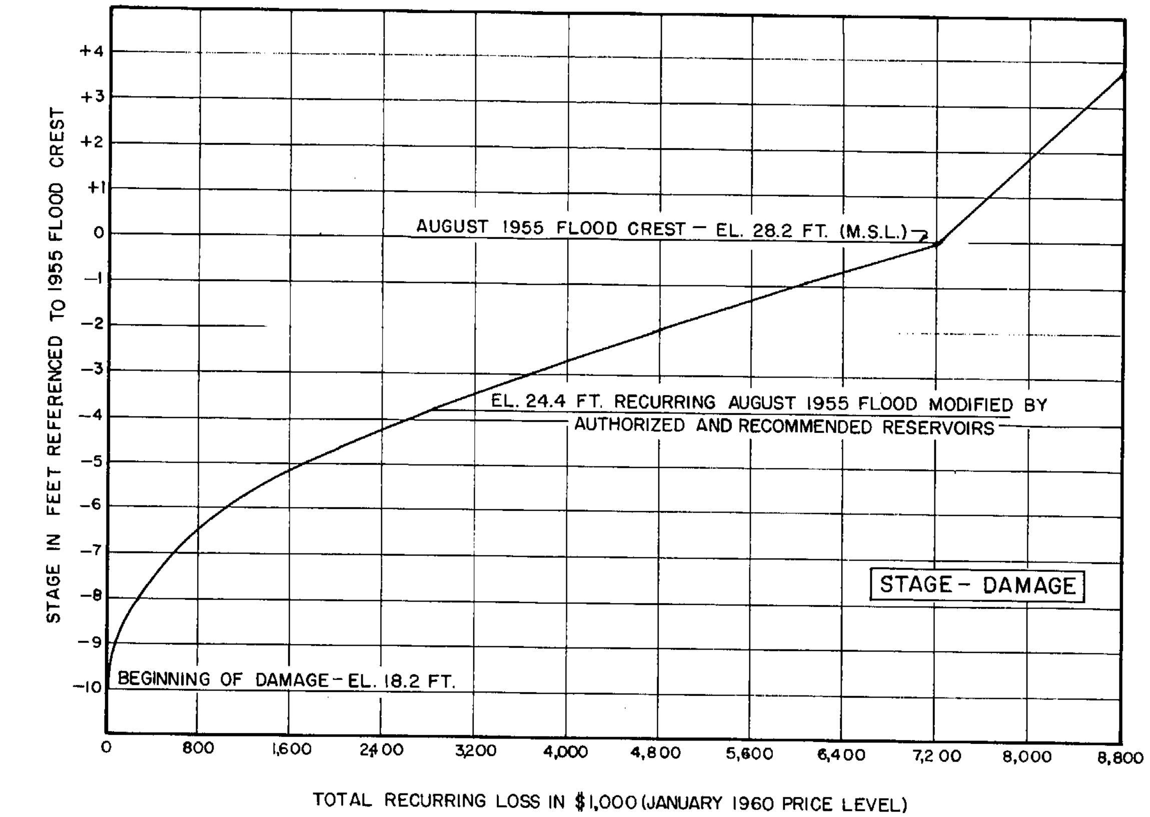


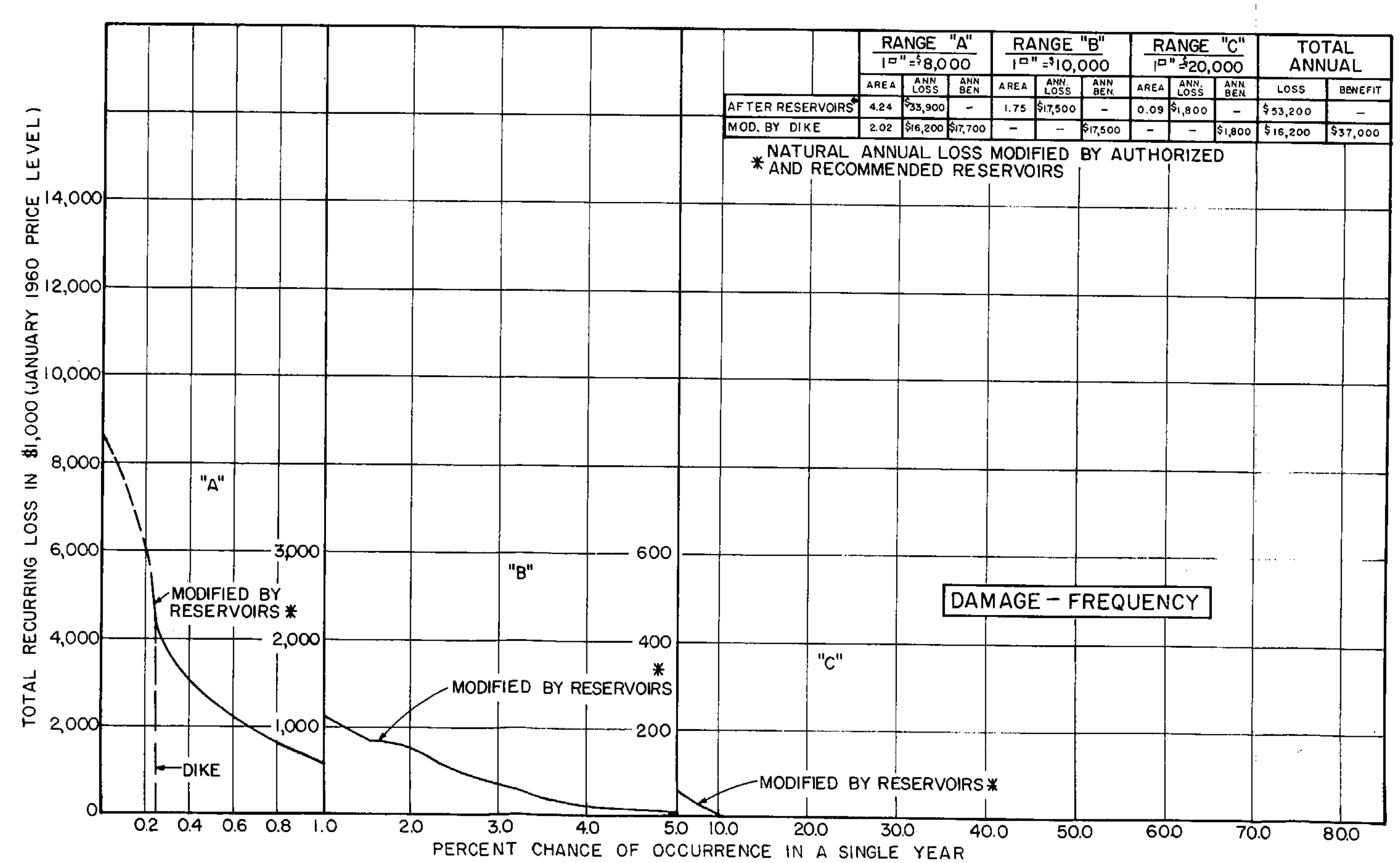


	RANGE "A" 10" = $2,000			RANGE "B"			RANGE "C"			TOTAL ANNUAL	
	AREA □"	ANN. LOSS	ANN. BEN.	AREA □"	ANN. LOSS	ANN. BEN.	AREA □"	ANN. LOSS	ANN. BEN.	LOSS	BENEFIT
AFTER RESERVOIRS ✱	4.75	$9,500	–	–	–	–	–	–	–	$9,500	–
MOD. BY DIKE	3.80	$7,600	$1,900	–	–	–	–	–	–	$7,600	$1,900

✱ NATURAL ANNUAL LOSS MODIFIED BY AUTHORIZED AND RECOMMENDED RESERVOIRS

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE-DAMAGE & DAMAGE-FREQUENCY CURVES
WEST BANK
UPSTREAM FROM MAPLE STREET
NAUGATUCK RIVER CONNECTICUT

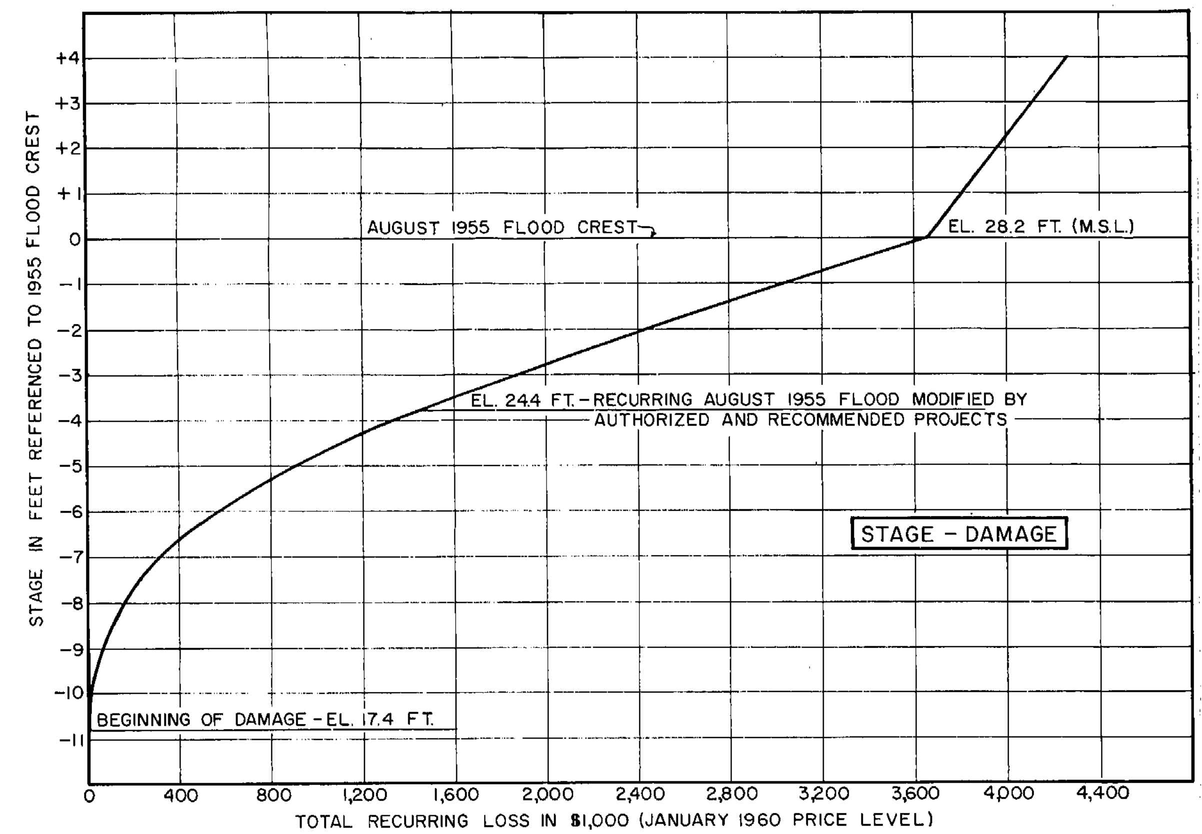


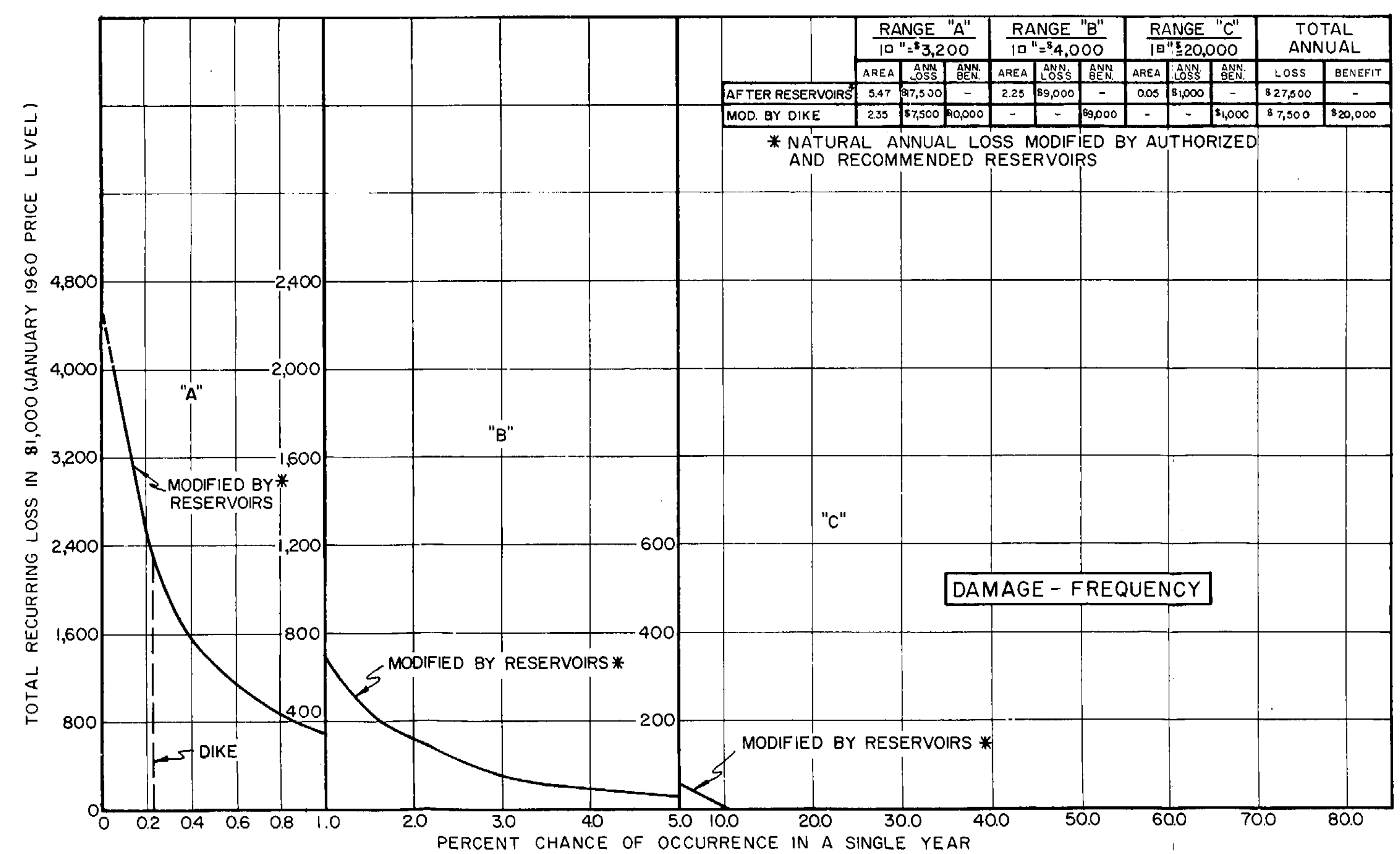


	RANGE "A" 1□" = $8,000			RANGE "B" 1□" = $10,000			RANGE "C" 1□" = $20,000			TOTAL ANNUAL	
	AREA	ANN LOSS	ANN BEN	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	LOSS	BENEFIT
AFTER RESERVOIRS*	4.24	$33,900	–	1.75	$17,500	–	0.09	$1,800	–	$53,200	–
MOD. BY DIKE	2.02	$16,200	$17,700	–	–	$17,500	–	–	$1,800	$16,200	$37,000

* NATURAL ANNUAL LOSS MODIFIED BY AUTHORIZED AND RECOMMENDED RESERVOIRS

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA – DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE-DAMAGE AND
FREQUENCY-DAMAGE CURVES
WEST BANK
BRIDGE STREET TO DIVISION STREET
NAUGATUCK RIVER CONNECTICUT





	RANGE "A" 1□"=$3,200			RANGE "B" 1□"=$4,000			RANGE "C" 1□"=$20,000			TOTAL ANNUAL	
	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	AREA	ANN. LOSS	ANN. BEN.	LOSS	BENEFIT
AFTER RESERVOIRS✱	5.47	$17,500	-	2.25	$9,000	-	0.05	$1,000	-	$27,500	-
MOD. BY DIKE	2.35	$7,500	$10,000	-	-	$9,000	-	-	$1,000	$7,500	$20,000

✱ NATURAL ANNUAL LOSS MODIFIED BY AUTHORIZED AND RECOMMENDED RESERVOIRS

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
STAGE-DAMAGE AND
DAMAGE FREQUENCY CURVES
WEST BANK
DOWNSTREAM FROM DIVISION STREET
NAUGATUCK RIVER CONNECTICUT

APPENDIX C

PROJECT DESCRIPTION AND COSTS

APPENDIX C

PROJECT DESCRIPTION AND COSTS

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EXHIBIT

APPENDIX C

PROJECT DESCRIPTION AND COSTS

1. GEOLOGY

Surficial geology and boring data for pile foundations for railroad and highway bridges indicate that bedrock will not be encountered in the required excavations. The only bedrock disclosed by reconnaissance occurs in the spillway of the hydroelectric plant upstream from Maple Street. Ground water will be encountered in excavations adjacent to the river at or near sea level and may be subject to minor tidal fluctuation. Materials to be excavated are indicated to be gravels and sands, the latter fine and silty in part. Additional gravelly materials, if needed for construction of the dikes, may be obtained from the river bed and/or adjacent areas to the west, near the upstream project limit. Till in the area, for use as impervious fill, is rather sandy. Possible sources have been located just north of State Route 34 in the southern part of Derby within a 3-mile haul and on the highland west of the Naugatuck River within a 5-mile haul. Existing riprap along the riverbank was obtained from a small quarry about two miles northeast of the site. The stone appears suitable for bank protection and an adequate supply for the project exists at the quarry. Other sources of suitable stone lie within a 15-mile haul. Several transit-mix concrete plants lie within a 10-mile haul of the site.

2. PROJECT DESCRIPTION

a. General. The studied local protection projects would provide for the construction of earth dikes and reinforced concrete flood walls along both banks of the Naugatuck River, a stream deflector in the river channel, pumping stations and appurtenant works, and the relocation of a portion of the river channel and a railroad spur track. The projects would provide protection against the standard project flood modified by authorized and recommended reservoirs. Profiles, general plans, and details of the recommended plan are shown on Plates C-1 to C-6 of this appendix. The most feasible plan, where it differs from the recommended plan, is shown on Plate C-7.

The study considered several plans for three separate areas of Ansonia and the adjacent portion of Derby along the Naugatuck River: the Division Street area on the west bank, for which four alternative plans were considered; the east bank, including the Beaver Brook area, for which one alternative plan was considered; and the River Street area on the west bank above Maple Street, for which one plan alternative to the most feasible plan and the recommended plan was considered.

b. Dike design. The dikes, in general, would have a top width of 10 feet with 1 on 2 side slopes. The major portion of the dikes would be constructed of random fill obtained from channel and storage pond excavations. A 4.5-to 5.5-foot thick, impervious blanket would be constructed within the dike. Where required, the river side of the dike would be faced with dumped rock on a 6-inch gravel bed, the thickness of the rock blanket varying from 1 foot to 3 feet, dependent upon anticipated channel velocities. Slopes not protected by dumped rock would be topsoiled and seeded. The project would be designed with a minimum of 3 feet of freeboard.

Excavated materials which are unsuitable for use in the dike or are in excess of the dike requirements could be spoiled in the area on the west bank between the railroad bridge and Division Street. Permission to use the spoil area would be obtained from the owners by the City of Ansonia.

c. Flood wall design. Flood walls are designed in accordance with criteria established in EM 1110-2-2501, modified as necessary to fit local conditions.

d. Most feasible plan.

(1) West bank.

(a) Division Street area. This plan would require about 6,200 feet of dike, having a maximum height of 30 feet above ground, 5 stoplog openings, and a pumping station, and would protect 104 acres of residential, industrial, and commercial properties, of which 62 are in Ansonia and 42 in Derby.

Starting in Derby at high ground formed by the embankment of Route 8 at a point about 500 feet beyond the Mill Street-Route 8 interchange, the dike would extend east to a stoplog structure at Mill Street, consisting of 2 openings respectively 12 and 16 feet high, separated by 20 feet of reinforced concrete wall. From there, the dike would extend to a 13-foot high stoplog crossing of the New Haven Railroad about 800 feet east of the starting point, and thence continue about 300 feet east before turning northeasterly to follow the west bank of the widened Naugatuck River to Division Street. A 9-foot high stoplog opening on the west ramp of the Division Street bridge would continue the protective works, followed by a dike forming the bank of the relocated river channel for 3,000 feet, extending westerly with a 17-foot stoplog opening at the railroad tracks, about 200 feet south of the railroad bridge and thence to high ground about 150 feet southeast of the Wooster Street-Clifton Avenue intersection. This alignment would also protect a large, vacant flood plain north of Division Street, including the site of the proposed municipal sewage treatment plant.

(b) River Street area. Protection for this area on the west bank would require the construction of 500 feet of reinforced concrete flood wall with 3 stoplog openings, 800 feet of dike, and a small pumping station with appurtenant structures, and would protect 10 acres of industrial land.

Beginning 150 feet north of the Maple Street-River Street intersection, a short dike section would run 160 feet to River Street, where a reinforced concrete flood wall 120 feet long would continue to a stoplog opening 13 feet in height, whence the wall extends along the edge of the roadway for 157 feet to a stoplog opening 14 feet in height. Beyond this the flood wall continues along River Street 100 feet to a stoplog opening, 16 feet high, and further extends beyond this opening to form the end wall of a dike which runs 650 feet to high ground, generally at right angles to both roadway and river, turning near the end to provide protection for a power transformer substation.

(2) East bank. Protection for the east bank would include 9,500 feet of dikes and flood walls and 5 stoplog structures, a stream deflector, 2 pumping stations with appurtenant structures, and 2 new bridges across Beaver Brook. Relocation of 1,600 feet of railroad spur track and a section of river channel would also be required. Protection would be afforded to 118 acres of residential, industrial, and commercial land on the east bank.

Beginning 45 feet north of a proposed new Central Street bridge over Beaver Brook, and continuing south of the new bridge with 400 feet of low gravity wall along the north bank of the widened and deepened brook, the protection generally follows the present alignment of Beaver Brook southwesterly with a dike which attains a maximum height of 16 feet at Main Street, crossing the north abutment of a proposed new Main Street bridge over the brook with a 14-foot high stoplog opening and following down to the Naugatuck River with a reinforced concrete wall at a maximum height of 31 feet, and thence tying into the main stream east bank protection.

Main stem protection begins with an earth dike at the Beaver Brook protective works and extends up the river side of the mill buildings formerly owned by the United Shoe Machinery Corp., almost wholly in the present river bed, necessitating a channel relocation below the railroad bridge. This 1,800-foot stretch of dike would be continued with a reinforced concrete wall 100 feet long and placed on top of the bank on the river side of a spur railroad track. After a 14-foot stoplog opening across this track, the wall extends 1,000 feet, with a maximum height of 17 feet, on the land side of the spur and main line of New Haven Railroad tracks, tying into the east abutment of the Bridge Street bridge. From the north side of this bridge abutment, a 14- to 16-foot high dike would extend northerly to a stoplog opening, 16 feet high, crossing the main line of the railroad and continuing north to the east bank pier of the Maple Street bridge as a reinforced concrete wall with a maximum height of 20 feet over a distance of 700 feet. From the bridge an L-shaped wall with a maximum height of 32 feet above ground, extending upstream about 1,200 feet parallel to the buildings of the Farrel-Birmingham Company, would connect with a 14-foot high T-type wall at a point about 240 feet downstream of the American Brass Company bridge. From a 13-foot high stoplog opening at the bridge, the wall would continue for approximately 1,060 feet upstream to a point where the clearance becomes sufficient to allow construction of a dike with a maximum height of 12 feet. The dike would extend 900 feet upstream, then turn away from the main channel in the vicinity of the American Brass Company hydroelectric plant, crossing the tracks of the New Haven Railroad, with a stoplog opening 7 feet high, and tying into the embankment of the company canal. A reinforced concrete wall 15 feet in height, having a 4-foot by 4-foot sluice gate in the center, would extend across this canal.

From the upstream end of the center pier of the New Haven Railroad bridge in Ansonia, a stream deflector would be constructed to aid in equalizing flows through the openings of the bridge by diverting water to the northerly opening. This should speed passage of flows under the bridge and prevent build-up of deposition under the northerly span.

(3) Channel work. Channel work for the Ansonia-Derby local protection project would consist of relocating and widening to insure a minimum width of 250 feet from the lower end of the project to just below the railroad bridge in Ansonia; channel clearing in the vicinity of the railroad bridge; and channel deepening and widening from immediately below the Bridge Street bridge to a point upstream of the American Brass Company private bridge.

e. Recommended plan. In the recommended plan, the west and east bank protection and the channel work described above under subparagraph d, "Most feasible plan," would remain the same except in the River Street area. The recommended plan in the River Street area, which is locally favored, includes 500 feet of reinforced concrete flood wall with 9 stoplog openings and 5 closure panels, 800 feet of dike, a small pumping station with appurtenant structures, and would protect the same 10 acres of industrial land as the most feasible plan.

As in the most feasible plan, the protective work in the River Street area would begin 150 feet north of the Maple Street-River Street intersection with a short dike section which would run 160 feet to River Street where a reinforced concrete flood wall 120 feet long would continue to a stoplog opening 13 feet in height. At the stoplog, an L-shaped reinforced concrete wall would run for 150 feet along the face of the office wing and manufacturing plant of the Ansonia Manufacturing Company. This wall has stoplog openings at 3 doors and 3 windows and closure panels at 5 windows. From the end of the L-shaped wall a reinforced concrete T-type wall would extend, at right angles, 19 feet to the edge of River Street and then turn up the edge of the roadway for a distance of 7 feet to a stoplog opening 14 feet in height. The T-type wall would continue along River Street 100 feet to a stoplog opening 16 feet high and further extend beyond this opening to form the end wall of a dike which would be as described in the most feasible plan. The recommended plan differs from the most feasible plan only along the front of the Ansonia Manufacturing Company buildings.

f. Other plans studied.

(1) West bank below Division Street. On the west bank below Division Street several alternative plans were investigated, none of which proved to be as economically feasible or as desirable to local interests.

In the first of these, the plan would provide protection for the Charlton Printing Company, the four acres of enhanceable land and a portion of railroad track, all in Derby, as in the recommended and most feasible plans, but would exclude from protection the drive-in theater, an enterprise protected in the most feasible plan. When considered with the recommended west bank plan above Division Street, this west bank plan has a total first cost of $2,010,000, annual charges amounting to $79,200 and annual benefits of $84,100. The benefit-cost ratio for this plan would be 1.1 to 1, which is less than for the recommended and most feasible plans.

In the second of these alternative plans below Division Street, the drive-in theater and the four acres of enhanceable land would be excluded. When this plan is considered with the most feasible plan above Division Street, it is found to have a total first cost of $1,910,000, annual charges amounting to $75,400, and annual benefits of $80,700. The benefit-cost ratio for this plan would be 1.1 to 1, which is less than that for the recommended and most feasible plans.

The third alternative plan in the area would exclude all of the properties below Division Street, in Derby, with the protective works extending west from the Naugatuck River up the north side of Division Street, thereby protecting only properties in Ansonia. Considered with the recommended plan above Division Street, this west bank plan would have a total first cost of $2,300,000, annual charges amounting to $90,100, and annual benefits of $62,200. The benefit-cost ratio of this plan is 0.7 to 1.

(2) West bank above Division Street. On the west bank above Division Street, one alternative plan was investigated. It considered an alignment continuing north of Division Street on the

recommended alignment for 600 feet and then swinging west, just north of the Hershey Metals, Inc., to the railroad embankment, then extending north up the river side of the tracks and meeting and continuing with the recommended alignment below the railroad bridge. This plan, when combined with the most feasible plan below Division Street, would have a total first cost of $1,700,000, annual charges of $67,100, and annual benefits of $60,400. The benefit-cost ratio of this plan would therefore be less than unity.

(3) East bank. On the east bank, one alternative plan was considered for the Beaver Brook protection. This plan included a concrete chute and an underground pressure conduit to carry Beaver Brook flows through the east bank dike to the river. The east bank dike would then be carried southeasterly to high ground. This Beaver Brook plan, when considered with the remaining main channel east bank plan, was found to have a total first cost of $4,810,000, annual charges of $181,700, and annual benefits of $192,200. This plan has a benefit-cost ratio of 1.1 to 1, which is less than for the recommended and most feasible plans.

(4) River Street area. In the River Street area a third plan, in addition to the most feasible and the recommended plans, was investigated. This plan included a wall, starting on the river side of the Maple Street-River Street intersection and continuing down the river side of River Street. The wall extends to the bottom of the channel and upstream along the river to where a stoplog structure crossing River Street would connect to the dike which, as in the recommended and most feasible plans, runs generally westerly, at right angles to River Street and the river, to high ground. The total first cost of this River Street area plan is $710,000, which is approximately double the cost of the recommended and most feasible plans.

(5) Channel work. Several plans of improvement were considered in various portions of the channel. Other than the recommended improvement, none proved feasible. Particular attention was given to the feasibility of replacing the present west stub abutment of the Maple Street bridge with a full abutment. Two plans were considered with the necessary river bank realignments and neither proved to be as economically feasible

as the comparatively minor improvement recommended, when considered economically and hydraulically with the entire project.

g. Pumping stations and drainage. Four pumping stations with appurtenant structures and collector systems would be required to handle interior drainage and seepage under the dikes and to provide for discharge of sanitary sewage from the city of Ansonia during periods of high water. Hydrologic and hydraulic design criteria for these stations are discussed in Appendix A. In the following paragraphs, the pumping stations in each of the areas to be protected are described.

(1) West bank. On the west bank, the Division Street pumping station for the discharge of sanitary sewage and storm drainage would be located downstream of the drive-in theater. Three 36-inch, axial flow, propeller pumps with right angle gear units and diesel engines would be housed in a structure 55 feet long by 60 feet wide. The pumps could discharge a total capacity of 102,000 gallons per minute of storm drainage at a head of 25 feet from the 1.3-acre storage pond, which would be fed by a system of interceptor and collector pipes and an open trench 1,100 feet long. Also, at this station, two 18-inch volute pumps, having a total pumping capacity of 4,400 gallons per minute at a like head, would be tied into the present sanitary sewer system of Ansonia and would discharge the entire sanitary sewage of the city during periods of high water. The pumps would be electric motor driven and could be operated by a standby generator in emergencies.

(2) East bank. Both pumping stations on the east bank would discharge only interior drainage. The Front Street station, located on Front Street about 150 feet west of Main Street, would have three 30-inch, axial flow, propeller pumps driven through right angle gear drives by diesel engines and housed in a structure 42 feet long by 22 feet wide. The pumps, with a total discharge capacity of 57,000 gallons per minute at a head of 30 feet from a storage pond with a surface area of 0.3 acres, would be fed by a system of collector and interceptor pipes to drain the area downstream of Maple Street. The Maple Street pumping station, located in the vicinity of the Maple Street bridge, would be housed in a building approximately 22 feet square and have three 18-inch 2-stage axial flow, propeller

pumps, driven by variable speed diesel engines, with a total pumping capacity of 23,000 gallons per minute with a head of 30.5 feet from an underground sump fed by a complete system of collector and interceptor pipes. This station would handle interior drainage and seepage under the dikes and flood walls above Maple Street.

(3) River Street area. In the River Street area, three 8-inch by 6-inch motor-driven pumps would be housed in an underground structure 19 feet long and 18 feet wide. This station would have a discharge capacity of 6,000 gallons per minute under a head of 27 feet from a sump and a complete system of interceptor and collector pipes to handle interior drainage and seepage under the dikes and flood walls.

3. EMBANKMENTS AND FOUNDATIONS

a. General. A program of investigation, consisting principally of field observations of the terrain, road cuts, river banks, and excavation areas and discussions with local inhabitants correlated with geological history, has been made to the extent considered necessary for this report, to determine (a) the characteristics of the foundation soils for the proposed embankment and concrete structures, (b) the characteristics of the materials to be excavated, (c) the availability and economics of sources of embankment materials, and (d) the characteristics of economically available embankment materials. Foundation explorations for the specific structures under study were limited to the excavation of 5 test pits and 1 test trench in the dike foundation area on the west bank between stations 330+00 and 362+00 to determine the character of the foundation fill and natural soils where the dike must be constructed for the greatest hydraulic heads. Foundation conditions were also estimated from logs of borings for bridge footings in the immediate area. The location and logs of selected bridge borings are shown on Plates C-1, C-2 and C-4 of this Appendix. The site geology of the area is described in paragraph 1 of this Appendix.

b. Characteristics of foundation soils. It is apparent from the preliminary investigations that none of the structures will be founded on bedrock and that, in general, the natural foundation soils for all structures consist of alluvial deposits of relatively pervious sand, sandy gravel and gravelly sand. Surface deposits of fill and trash exist in a large portion of the foundation areas. On the west side of the river, between stations 330/00 and 355/00, there is a sandy gravel fill for a depth up to 6 feet and between stations 342/00 and 355/00 there is a riverside spoil dike (from Disaster Relief Operations) about 15 feet high, composed of loose, sandy gravel. For this same dike foundation area, there is a surface deposit of trash up to 10 or 15 feet deep between stations 355/00 and 362/00. This portion of the foundation area forms part of the city dump. The surficial deposits on the east side of the river between stations 190/00 and 250/00 are mainly man-made fills of pervious sand and gravelly sand, with variable depth but generally less than 5 feet. In the foundation area for the longer dike on the west side of the river in the River Street area, there is a surface fill deposit, about 10 feet in thickness, composed of sand containing a large percentage of coal clinkers and ashes.

c. Characteristics of excavated materials. All the soils to be excavated for the construction of the dike and concrete structures and for channel improvements are sand, sandy gravel and gravelly sand except for minor amounts of trash, fill containing ashes and trash, and stripping material. In general, the materials to be excavated will be pervious and can be used in the random portion of the dike embankments. It is estimated that a sufficient quantity of random embankment material will be obtained from the required excavation to construct all dikes. Excess and unsuitable excavation may be spoiled in the area west of the dike on the west side of the river between stations 365/00 and 386/00.

d. Characteristics of available embankment materials. It is expected that a sufficient quantity of random embankment material which will be generally pervious sand, gravelly sand and sandy gravel will be obtained from the required excavations. Similar granular materials can be obtained, if necessary, from sources in the river valley near the limits of the project. As described in paragraph 1 of this appendix, there are several deposits of glacial till within an average haul of five miles which contain soils considered suitable for the impervious section of the dike embankments. The glacial tills are well graded, gravelly, silty sand containing more than 35 percent silt size. For the purpose of this report, it is assumed that impervious fill material will be obtained just off State Route 34, south of the project, which will necessitate an average haul of three miles over heavy duty roads and partially through a heavily populated area. Processed gravel is available from commercial sources within a mile of the work. Rockfill is available from sources described in paragraph 1.

e. Foundation design for concrete structures. The preliminary designs for the stoplog structures and pumping stations shown in the plans are for pervious granular foundation conditions, assuming little, if any, foundation settlement. The flood wall sections shown on the plan are based on a design assuming a maximum allowable foundation pressure of two tons per square foot which is considered a satisfactory criterion for a granular foundation. Prior to final design of the walls, detailed exploration must be made to determine the characteristics and depth of the surficial fill deposits in the foundation areas. For the preliminary design it is assumed that, in general, a satisfactory foundation soil occurs at a depth of four feet below the existing ground surface. The control of foundation seepage will be a major item in the final design of the walls. For the wall sections shown on the plans, the creep ratios are greater than the minimum permissible values shown in Engineering Manual 1110-2-2501, "Flood Walls," page 14, for sand and sandy gravel foundation soils. Additional foundation seepage control is provided by the landside foundation toe drain, including a drain pipe extending to a pumping station. Considering the highly pervious nature of the foundation soils, it is roughly estimated that the rate of foundation seepage will be in the order of 0.08 gallon per minute foot of hydrostatic head per lineal foot of wall. This estimate is based on an assumed coefficient of permeability of .01 cm/sec for the foundation soils.

f. Design of earth dikes. The selected sections for the various reaches of the dikes are shown on Plates C-2 to C-7 of this Appendix. These sections were selected to utilize to the maximum extent possible the sand, gravelly sand, and sandy gravel soils available from required excavations and nearby deposits and to minimize the use of the more costly impervious fill. The selected slopes are considered adequate to provide stable embankments, considering the characteristics of the embankment and foundation soils and relatively small heights of the embankments. Stripping is limited to the removal of trash in the foundation area within the limits of the city dump and the removal of the existing dike south of Division Street. The most critical design feature of the dikes is the control of seepage through the embankments and the pervious foundations. Seepage is controlled in part by providing an embankment consisting generally of a riverside impervious fill section of glacial till and a landside random fill section of sand, gravelly sand and sandy gravel. For the dike along the east side of the channel between stations 172/00 and 190/00, the impervious section has been located to permit separate construction of the cofferdam and riverside random section and to reduce possible detrimental effects in the impervious section from settlement of the uncompacted cofferdam.

Foundation cutoffs are considered impractical for control of foundation seepage because it is apparent that impervious soils are not present in the foundations within reasonable depths. Partial foundation cutoffs, however, are provided for the dikes immediately adjacent to the river channel by extending the impervious sections to the bottom of the channel. For the preliminary design of dike on fill north of Bridge Street a partial foundation cutoff is provided through the existing fill since the character of the fill has not been determined by explorations. A cutoff has been provided for the dike on the west side of the river between stations 330/00 and 342/00, extending through the possible stratum of organic silt to insure against movement of the silt particles by seepage forces.

Along the entire length of all dikes, a landside foundation drain, including a perforated drain pipe extending to a pumping station, has been provided to insure against the development of detrimental seepage uplift pressures. It is considered that pumping from these toe drains, along with the impervious features provided, will adequately control seepage. The coefficient of permeability of the foundation soils is estimated to be in the order of 0.01 cm/sec. The rate of seepage through the foundation is roughly estimated to be in the order of 0.08 gallons per minute per foot of hydrostatic head per lineal foot of dike. Rockfill on gravel bedding is provided on all riverside slopes, where necessary, for protection against erosion by the river. The thickness of rockfill was based on riverflow velocities at the various points. Gravel bedding is provided as a transition between the rockfill and the embankment material. The landside slopes of dikes are to be dressed with seeded topsoil to provide protection against erosion from wind and rain.

4. COST ESTIMATES

a. Basis of estimates. Topographic maps of the U. S. Army Map Service, to a scale of 1:25,000 with 20-foot contours, were supplemented by plane table topographic and planimetric surveys of the studied dike alignments and adjacent topography. Foundation conditions were determined by field reconnaissance, test pits, and examination of logs of borings for bridge foundations. Quantities of the principal construction items were estimated on the basis of a preliminary design which would provide safe and adequate structures. Hydrologic and hydraulic criteria adopted for the design of dikes, flood walls, pumping stations, and channel relocation are discussed in Appendix A. Local interests have indicated that an area on the west bank between the railroad bridge and Division Street would be available for spoiling excavated materials in excess of dike requirements. The location of this spoil area is shown on Plate C-3 of this Appendix. Benefits attributable to the studied plans are discussed in Appendix B. A breakdown of first costs for both the most feasible and the recommended projects is given in Table C-1 at the end of this Appendix.

b. Unit prices. Unit prices are based on average bid prices, adjusted to January 1960 price levels, for similar projects in the New England area. Costs of lands and improvements required for the studied projects are based on information from local officials and reflect values in recent sales in the area to which have been added acquisition costs.

c. Contingencies, engineering and overhead. To cover contingencies, construction and relocation costs have been increased 20 percent. Costs of preauthorization studies, engineering and design, and supervision and administration are estimated lump sums based on knowledge of the project areas and experience on similar projects.

d. Apportionment of costs. Bureau of the Budget Circular A-47, dated 31 December 1952, provides, in part, that there shall be a payment or contribution by local interests toward the total cost of a project equal to 50 percent of the amount determined by applying to the total cost of the project the ratio of land enhancement benefits to total benefits. Since local interests will pay for all lands, damages, and relocations for the entire project, a proportionate reduction in their cash contribution is allowed. The derivation of non-Federal cash contribution for the most feasible plan is shown in Table C-2 at the end of this appendix. Table C-3 summarizes the financial and economic costs for the most feasible projects; Table C-4 summarizes investment costs.

e. Annual charges. The estimate of Federal annual charges includes interest at 2.5 percent on the Federal investment plus the amount required to amortize the investment over the assumed 50-year life of the project. The investment represents the Federal first cost plus interest during the estimated 2-year construction period. Non-Federal interest and amortization charges were computed in a similar manner. Recent experience of the City of Ansonia in the sale of municipal bonds, as shown in letter from the City Corporation Counsel attached to this appendix, indicates an average interest rate of 2.5 percent. However, an interest rate of 3 percent, 0.5 percent higher than the rate applied in determining Federal charges, was used since the Federal financing rate should be considered a risk floor from which all other forms of investment must be measured, and the gap of 0.5 percent between Federal and non-Federal rates is believed to be proportionate to the real differential in return from Federal and from tax-free municipal bonds viewed over a long period of evaluation such as the period of project amortization. Non-Federal charges also include amounts for maintenance and operation of the project and for interim replacement of equipment having an estimated life of less than 50 years. No allowance is made for tax loss on lands removed from taxation since the amount of such loss will be more than offset by the increase in the value of the protected properties. The derivation of annual charges is given in Table C-5.

f. Recommended plan.

(1) Apportionment of costs. Local interests would be required and have agreed to pay the additional cost between the recommended plan and the most feasible plan. The apportionment of costs for the most feasible plan is therefore the basis for apportioning costs in the recommended plan. This difference is now estimated to be $80,000, and would make the local cash contribution for construction $850,000, or 14.9 percent of the total cost of construction. A summary of the financial and economic costs for the recommended plan is given in Table C-6. Table C-7 summarizes investment costs.

(2) Interest and amortization. The rates of interest, both Federal and non-Federal, and the period for amortization for the recommended plan are the same as those used in the most feasible plan. Table C-8 shows the derivation of annual charges.

TABLE C-1

FIRST COST - ANSONIA-DERBY LOCAL PROTECTION
(January 1960 Price Level)

MOST FEASIBLE AND RECOMMENDED PROJECTS

Item	Unit Price	Unit	Most Feasible Plan		Recommended Plan	
			Quantity	Amount	Quantity	Amount
Lands and Damages						
Lands and Easements		L.S.		$ 78,000		$ 78,000
Improvements		L.S.		42,000		42,000
Total Lands etc.				120,000		120,000
Relocations						
Sanitary Sewer						
24" C.I. Conc. Incased Pipe	$ 30.00	L.F.	765	22,950	765	22,950
24" R.C. Pipe	8.50	L.F.	3,560	30,260	3,560	30,260
Regular Manholes	200.00	Ea.	4	800	4	800
Special Manholes (3)		L.S.		4,800		4,800
Catwalk to Manhole		L.S.	1	2,500	1	2,500
24" Gate Valve	1,000.00	Ea.	2	2,000	2	2,000
24" Tide Gate	400.00	Ea.	1	400	1	400
Central St. Bridge						
Structural Steel	400.00	Ton	8	3,200	8	3,200
Concrete, (Reinforced)	80.00	C.Y.	126	10,080	126	10,080
Grate Deck Steel	45.00	S.Y.	200	9,000	200	9,000
Approach Paving	10.00	S.Y.	200	2,000	200	2,000
Sidewalks (8' Wide)	25.00	L.F.	120	3,000	120	3,000
Guard Rail	5.00	L.F.	60	300	60	300
Main St. Bridge						
Structural Steel	400.00	Ton	8	3,200	8	3,200
Concrete, Reinforced	80.00	C.Y.	200	16,000	200	16,000
Paving	10.00	S.Y.	400	4,000	400	4,000
Sidewalks (8' Wide)	25.00	L.F.	120	3,000	120	3,000
Guard Rail	5.00	L.F.	60	300	60	300

TABLE C-1 (Cont'd)

Item	Unit Price	Unit	Most Feasible Plan Quantity	Most Feasible Plan Amount	Recommended Plan Quantity	Recommended Plan Amount
Railroad	$ 5.00	L.F.	1,600	$ 8,000	1,600	$ 8,000
Contingencies				26,210		26,210
Engineering & Design				15,000		15,000
Supervision & Administration				13,000		13,000
Total Relocations				180,000		180,000
Total Lands and Relocations				300,000		300,000
Construction						
Channels & Canals						
Excavation, (Common)	0.50	C.Y.	354,000	177,000	354,000	177,000
Excavation, Stream Deflector	15.00	C.Y.	200	3,000	200	3,000
Pumping, Stream Deflector		L.S.		5,000		5,000
Concrete, Stream Deflector	50.00	C.Y.	1,300	65,000	1,300	65,000
Concrete, (Rein.)	55.00	C.Y.	1,000	55,000	1,000	55,000
Concrete, (Mass.)	40.00	C.Y.	500	20,000	500	20,000
Rock Borrow & Place	5.00	C.Y.	5,350	26,750	5,350	26,750
Gravel Bedding	1.00	C.Y.	1,050	1,050	1,050	1,050
Steel Sheeting (Temp.)	2.00	S.F.	3,100	6,200	3,100	6,200
Contingencies		L.S.		72,000		72,000
Engineering & Design		L.S.		43,000		43,000
Supervision & Administration		L.S.		38,000		38,000
Total Channels & Canals				512,000		512,000
Levees & Floodwalls						
Site Preparation	100.00	ACS	54.5	5,450	54.5	5,450
Stream Control		L.S.		40,000		40,000
Building Removal		L.S.		14,000		14,000
Concrete Removal		L.S.		3,500		3,500
R.R. Track Removal		L.S.		2,000		2,000
R.R. Trestle Removal		L.S.		2,000		2,000
Excavation, Common	0.50	C.Y.	26,500	13,250	26,500	13,250

TABLE C-1 (Cont'd)

Item	Unit Price	Unit	Most Feasible Plan Quantity	Most Feasible Plan Amount	Recommended Plan Quantity	Recommended Plan Amount
Excavation, Structure	$ 0.70	C.Y.	166,250	$ 116,375	166,550	$ 116,585
Rock, Placing	1.00	C.Y.	34,000	34,000	34,000	34,000
Coffer Dam, Placing	0.30	C.Y.	51,500	15,450	51,500	15,450
Embankment, Placing	0.35	C.Y.	401,700	140,595	401,700	140,595
Backfill	0.20	C.Y.	22,540	4,508	22,540	4,508
Impervious, Borrow	0.85	C.Y.	127,700	108,545	127,700	108,545
Pervious, Borrow	1.00	C.Y.	21,350	21,350	21,350	21,350
Gravel, Select, Borrow	3.00	C.Y.	12,250	36,750	11,900	35,700
Gravel, Common, Borrow	1.00	C.Y.	7,000	7,000	7,000	7,000
Rock, Borrow	4.00	C.Y.	29,600	118,400	29,600	118,400
Concrete, Mass.	40.00	C.Y.	1,280	51,200	1,280	51,200
Concrete, Reinforced	100.00	C.Y.	0	0	558	55,800
" "	55.00	C.Y.	21,153	1,163,415	20,942	1,151,810
Steel Sheeting, Permanent	3.00	S.F.	86,440	259,320	86,629	259,887
Steel Sheeting, Temporary	2.00	S.F.	67,900	135,800	67,900	135,800
Loaming, (4"), Borrow & Place	0.50	S.Y.	61,650	30,825	61,650	30,825
Seeding	0.35	S.Y.	61,650	21,578	61,650	21,578
Aluminum Stop Logs	25.00	S.F.	4,880	122,000	5,062	126,550
Misc. Hardware		L.S.		0		8,500
Stoplog Storage Shelters	1.00	C.F.	13,100	13,100	13,250	13,250
Misc. Paving		L.S.		37,000		37,000
4'x4' Sluice Gate	1,000.00	Ea.	1	1,000	1	1,000
6' Woven Wire Fence	4.00	L.F.	1,700	6,800	1,700	6,800
Manholes, Regular	200.00	Ea.	30	6,000	30	6,000
Manholes, Special	1,000.00	Ea.	28	28,000	28	28,000
Catch Basin	200.00	Ea.	21	4,200	21	4,200
Tailrace Manhole	2,000.00	Ea.		2,000		2,000
Drainage Pipe						
8" C.M.	3.00	L.F.	5,180	15,540	5,180	15,540
12" C.M.	3.50	L.F.	9,650	33,775	9,650	33,775
15" C.M.	4.50	L.F.	675	3,038	675	3,038
18" C.M.	5.50	L.F.	290	1,595	290	1,595

TABLE C-1 (Cont'd)

Item	Unit Price	Unit	Most Feasible Plan Quantity	Most Feasible Plan Amount	Recommended Plan Quantity	Recommended Plan Amount
21" C.M.	$ 6.00	L.F.	380	$ 2,280	380	$ 2,280
24" C.M.	7.50	L.F.	320	2,400	320	2,400
12" C.C.	4.00	L.F.	1,325	5,300	1,325	5,300
12" R.C.	4.50	L.F.	280	1,260	280	1,260
15" R.C.	5.00	L.F.	660	3,300	660	3,300
18" R.C.	6.00	L.F.	820	4,920	820	4,920
21" R.C.	6.50	L.F.	430	2,795	430	2,795
24" R.C.	8.50	L.F.	2,715	23,078	2,715	23,078
30" R.C.	9.50	L.F.	1,775	16,863	1,775	16,863
36" R.C.	12.00	L.F.	3,040	36,480	3,040	36,480
48" R.C.	20.00	L.F.	870	17,400	870	17,400
54" R.C.	30.00	L.F.	4,350	130,500	4,350	130,500
60" R.C.	35.00	L.F.	280	9,800	280	9,800
Contingencies		L.S.		576,265		586,873
Engineering & Design		L.S.		346,000		352,000
Supervision & Administration		L.S.		304,000		310,000
Total Levees & Floodwalls				4,112,000		4,192,000
Pumping Plants						
Division Street Station						
Storm Station						
Pumps & Drives		L.S.		110,000		110,000
Superstructure	2.00	C.F.	41,160	82,320	41,160	82,320
Substructure	60.00	C.Y.	309	18,540	309	18,540
Excavation	0.90	C.Y.	1,140	1,026	1,140	1,026
Accessories		L.S.		45,900		45,900
Sanitary Station						
Pumps & Drives		L.S.		25,400		25,400
Superstructure	2.00	C.F.	31,878	63,756	31,878	63,756
Substructure	60.00	C.Y.	490	29,400	490	29,400

TABLE C-1 (Cont'd)

Item	Unit Price	Unit	Most Feasible Plan Quantity	Most Feasible Plan Amount	Recommended Plan Quantity	Recommended Plan Amount
Excavation	$ 0.90	C.Y.	2,121	$ 1,909	2,121	$ 1,909
Accessories		L.S.		12,400		12,400
Front Street Station						
Pumps & Drives		L.S.		46,000		46,000
Superstructure	2.00	C.F.	26,568	53,136	26,568	53,136
Substructure	60.00	C.Y.	211	12,660	211	12,660
Excavation	0.90	C.Y.	857	771	857	771
Accessories		L.S.		46,010		46,010
Maple Street Station						
Pumps & Drives		L.S.		23,700		23,700
Superstructure	2.00	C.F.	13,202	26,404	13,202	26,404
Substructure	60.00	C.Y.	222	13,320	222	13,320
Excavation	0.90	C.Y.	775	698	775	698
Accessories		L.S.		20,480		20,480
River Street Station						
Pumps & Drives		L.S.		4,500		4,500
Substructure	60.00	C.Y.	54	3,240	54	3,240
Excavation	0.90	C.Y.	135	122	135	122
Accessories		L.S.		18,700		18,700
Contingencies		L.S.		132,608		132,608
Engineering & Design		L.S.		79,000		79,000
Supervision & Administration		L.S.		70,000		70,000
Total Pumping Plants				942,000		942,000
Model Study						
Construction & Testing		L.S.		50,000		50,000
Supervision & Administration		L.S.		4,000		4,000
Total Construction				5,620,000		5,700,000
Total Lands and Relocations (from page C-17)				300,000		300,000
Preauthorization Studies		L.S.		20,000		20,000
Total First Cost				5,940,000		6,020,000

TABLE C-2

APPORTIONMENT OF COSTS - MOST FEASIBLE PLAN
ANSONIA-DERBY LOCAL PROTECTION

1. Project First Costs

Construction	$5,620,000*
Lands, etc.	300,000
Total	$5,920,000

2. Project Benefits

Flood damage prevention	206,000	=	71.0%
Enhancement	84,000	=	29.0%
Total	$ 290,000	=	100.0%

3. Classification of Project Costs

	Flood Prev. (71.0%)	Enhancement (29.0%)	Total (100%)
Construction	$3,993,000	$1,627,000	$5,620,000
Lands, etc.	213,000	87,000	300,000
Total	$4,206,000	$1,714,000	$5,920,000

4. Computation of Non-Federal Share

	Tentative Apportionment		Adjusted Apportionment	
	Federal	Non-Federal	Federal	Non-Federal
Flood Prev. Portion				
Construction	$3,993,000	$ -	$3,993,000	$ -
Lands, etc.	-	213,000	-	213,000
Total	3,993,000	213,000	3,993,000	213,000
Land Enhancement Portion				
Construction	813,500	813,500	857,000	770,000
Lands, etc.	43,500	43,500	-	87,000
Total	857,000	857,000	857,000	857,000
Total First Cost				
Construction			4,850,000	770,000
Lands, etc.			-	300,000
Total			$4,850,000	$1,070,000

Based on the above apportionment, non-Federal interests, in addition to furnishing lands, etc., would have to make a cash contribution of $770,000 for the most feasible plan.

*Exclusive of preauthorization costs of $20,000.

TABLE C-3

ANSONIA-DERBY, CONN.
MOST FEASIBLE PLAN

SUMMARY OF PROJECT COSTS

Item	Financial Costs	Economic Costs
a. Federal project costs:		
(1) Project costs - Corps of Engineers:		
(a) Total project construction costs, including engineering, overhead and contingency allowances; and preauthorization survey costs	$5,640,000	$5,640,000
(b) Estimated market value of lands, easements, and rights-of-way not to be provided by others	-	0
(c) Corps of Engineers' project construction costs, excluding market value of land	-	5,640,000
(d) Non-Federal cash contribution toward construction	770,000	770,000
(e) Project net costs, Corps of Engineers	4,870,000	4,870,000
(f) Present worth of future additions for project purposes	-	0
(g) Total Corps of Engineers' project net costs for economic evaluation	-	4,870,000

TABLE C-3 (Cont.)

SUMMARY OF PROJECT COSTS

Item	Financial Costs	Economic Costs
. Federal project costs (cont.)		
(2) Project costs - other Federal Agencies	$ 0	$ 0
(3) Total Federal project costs	4,870,000	4,870,000
b. Non-Federal project costs:		
(1) Total project construction costs, including lands, easements, and rights-of-way; alteration, relocation, reconstruction or removal of highways, bridges, utilities, buildings, drainage facilities, etc.; engineering, overhead, and contingency allowances; land acquisition costs	300,000	300,000
(2) Estimated market value of lands, easements and rights-of-way to be furnished by non-Federal interests	-	120,000
(3) Non-Federal project construction costs, excluding market value of land	-	180,000
(4) Cash contribution toward construction to be provided by non-Federal interests as a special requirement of local cooperation	770,000	770,000
(5) Non-Federal project net costs	1,070,000	1,070,000
(6) Present worth of future additions required for project purposes	-	0
(7) Total non-Federal project net costs for economic evaluation	-	1,070,000
c. Total project first costs:	$5,940,000	$5,940,000

TABLE C-4

ANSONIA-DERBY, CONN.
MOST FEASIBLE PLAN

INVESTMENT COSTS

Item	Financial Investment	Economic Investment
a. Federal investment:		
(1) Recapitulation of project costs:		
Total Federal project net costs	$4,870,000	$4,870,000
(2) Interest during construction		
Basic interest @2.5% on total Federal project net costs	122,000	122,000
(3) Total Federal investment	4,992,000	4,992,000
b. Non-Federal investment		
(1) Recapitulation of project costs		
(a) Total non-Federal project net costs, including lands and cash contribution	1,070,000	1,070,000
(b) Estimated market value of lands provided by non-Federal interests	-	78,000
(2) Interest during construction		
(a) Basic interest @ 3.0% on total non-Federal project net costs	32,000	32,000
(b) Adjustment for any net loss in productivity of lands during construction	-	0
(c) Total interest during construction	-	32,000
(3) Total non-Federal gross investment	1,102,000	1,102,000
(4) Net salvage value of any non-Federally owned portion of the project at the end of the assumed amortization period for the project as a whole	-	0
(5) Total non-Federal net investment	1,102,000	1,102,000
c. Total net investment costs	$6,094,000	$6,094,000

TABLE C-5

ANSONIA-DERBY, CONN.
MOST FEASIBLE PLAN

ANNUAL COSTS

Item	Financial Costs	Economic Costs
a. Federal annual costs:		
(1) Interest on investment (2-1/2%)	$ 124, 800	$124, 800
(2) Amortization of net investment (1, 026%)	51, 200	41, 200
(3) Total Federal annual costs	176, 000	176, 000
b. Non-Federal annual costs:		
(1) Interest on gross investment (3. 0%)	33, 000	33, 000
(2) Adjustment for net loss of productivity on land $78, 000x(6%-3%)	-	2, 300
(3) Total economic interest and productivity loss on gross non-Federal investment	-	35, 300
(4) Amortization of net investment (0. 887%)	9, 800	9, 800
(5) Maintenance and operation, average annual non-Federal costs of maintenance and operation	5, 500	5, 500
(6) Allowance for major replacements	5, 200	5, 200
(7) Loss of taxes	-	0
(8) Total non-Federal annual costs	53, 500	55, 800
c. Total annual costs:	$ 229, 500	$ 231, 800

TABLE C-6

ANSONIA-DERBY, CONN.
RECOMMENDED PLAN

SUMMARY OF PROJECT COSTS

Item	Financial Costs	Economic Costs
a. Federal project costs:		
(1) Project costs - Corps of Engineers:		
(a) Total project construction costs, including engineering, overhead and contingency allowances; and preauthorization survey costs	$5,720,000	$5,720,000
(b) Estimated market value of lands, easements, and rights-of-way not to be provided by others	-	0
(c) Corps of Engineers' project construction costs, excluding market value of land	-	5,720,000
(d) Non-Federal cash contribution toward construction	850,000	850,000
(e) Project net costs, Corps of Engineers	4,870,000	4,870,000
(f) Present worth of future additions for project purposes	-	0
(g) Total Corps of Engineers' project net costs for economic evaluation	-	4,870,000
(2) Project costs - other Federal Agencies	-	0
(3) Total Federal project costs	4,870,000	4,870,000

TABLE C-6 (Cont.)

SUMMARY OF PROJECT COSTS

Item	Financial Costs	Economic Costs
b. Non-Federal project costs:		
(1) Total project construction costs, including lands, easements, and rights-of-way; alteration, relocation, reconstruction or removal of highways, bridges, utilities, buildings, drainage facilities, etc.; engineering, overhead, and contingency allowances; land acquisition costs	$ 300,000	$ 300,000
(2) Estimated market value of lands, easements and rights-of-way to be furnished by non-Federal interests	-	120,000
(3) Non-Federal project construction costs, excluding market value of land	-	180,000
(4) Cash contribution toward construction to be provided by non-Federal interests as a special requirement of local cooperation	850,000	850,000
(5) Non-Federal project net costs	1,150,000	1,150,000
(6) Present worth of future additions required for project purposes	-	0
(7) Total non-Federal project net costs for economic evaluation	-	1,150,000
c. Total project first costs	$6,020,000	$6,020,000

TABLE C-7

ANSONIA-DERBY, CONN.
RECOMMENDED PLAN

INVESTMENT COSTS

Item	Financial Investment	Economic Investment
a. Federal investment:		
(1) Recapitulation of project costs:	$4,870,000	$4,870,000
(2) Interest during construction Basic interest @ 2.5% on total Federal project net costs	122,000	122,000
(3) Total Federal investment	4,992,000	4,992,000
b. Non-Federal investment		
(1) Recapitulation of project costs		
(a) Total non-Federal project net costs, including lands and cash contribution	1,150,000	1,150,000
(b) Estimated market value of lands provided by non-Federal interests	-	78,000
(2) Interest during construction		
(a) Basic interest @ 3.0% on total non-Federal project net costs	35,000	35,000
(b) Adjustment for any net loss in productivity of lands during construction	-	0
(c) Total interest during construction	-	35,000
(3) Total non-Federal gross investment	1,185,000	1,185,000
(4) Net salvage value of any non-Federally owned portion of the project at the end of the assumed amortization period for the project as a whole	-	0
(5) Total non-Federal net investment	1,185,000	1,185,000
c. Total net investment costs	$6,177,000	$6,177,000

TABLE C-8

ANSONIA-DERBY, CONN.
RECOMMENDED PLAN

ANNUAL COSTS

	Item	Financial Costs	Economic Costs
a.	Federal annual costs:		
	(1) Interest on investment (2-1/2%)	$124,800	$124,800
	(2) Amortization of net investment (1.026%)	51,200	51,200
	(3) Total Federal annual costs	176,000	176,000
b.	Non-Federal annual costs:		
	(1) Interest on gross investment (3.0%)	35,600	35,600
	(2) Adjustment for net loss of productivity on land $78,000x(6%-3%)	-	2,300
	(3) Total economic interest and productivity loss on gross non-Federal investment	-	37,900
	(4) Amortization of net investment (0.887%)	10,500	10,500
	(5) Maintenance and operation, average annual non-Federal costs of maintenance and operation	5,500	5,500
	(6) Allowance for major replacements	5,200	5,200
	(7) Loss of taxes	-	0
	(8) Total non-Federal annual costs	56,800	59,100
c.	Total annual costs:	$232,800	$235,100

JOSEPH B. BUCKLEY
ATTORNEY AT LAW
156 MAIN STREET
ANSONIA, CONNECTICUT
TEL. REGENT 5-2220

February 23, 1959

General Alden K. Sibley
Division Engineer
U.S. Army Corp of Engineers
424 Trapelo Road
Waltham 54, Mass.

ATTN: William Slagle

Gentlemen:

The telephone request of February 18, 1959 from your Mr. William Slagle to Mr. Tom T. Wuerth of the Ansonia, Connecticut Redevelopment Agency for information concerning the municipal bonds which have been and may be issued by the City of Ansonia has been referred to me for reply.

The City of Ansonia is limited under ordinary circumstances to issue its bonds only for twenty years.

The city last issued bonds in 1957 at which time the rate of interest paid was 2.4 and a small premium. The interest rate varies as the bond market goes up or down. In 1957 the amount of the bond issue was $114,000.00. In 1956 a bond issue of $310,000.00 was sold at an interest rate of 2.9 %. In 1956 bonds in the amount of $27,000.00 were sold at an interest rate of 2.25 %. In 1955 bonds in the amount of $340,000.00 were sold at an interest rate of 2.2%.

I hope that the above has furnished the information you were seeking.

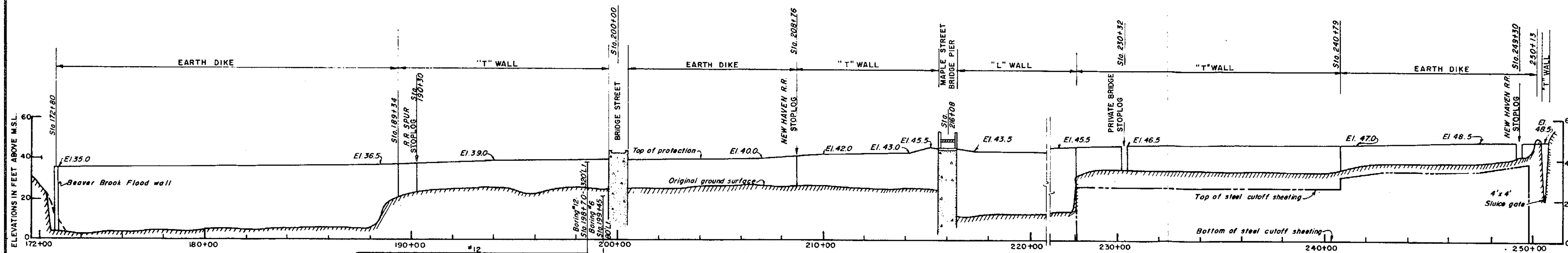
Very truly yours,

Joseph B. Buckley

Joseph B. Buckley
Corporation Counsel
City of Ansonia

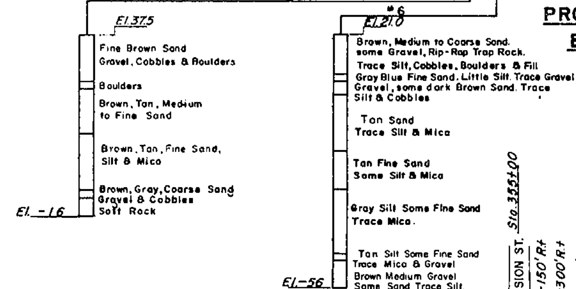
EXHIBIT C-1

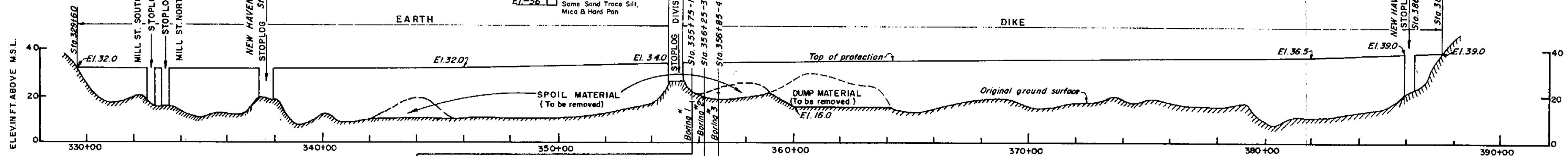
CORPS OF ENGINEERS U. S. ARMY



PROFILE ALONG ℄ OF PROTECTION
EAST BANK NAUGATUCK RIVER

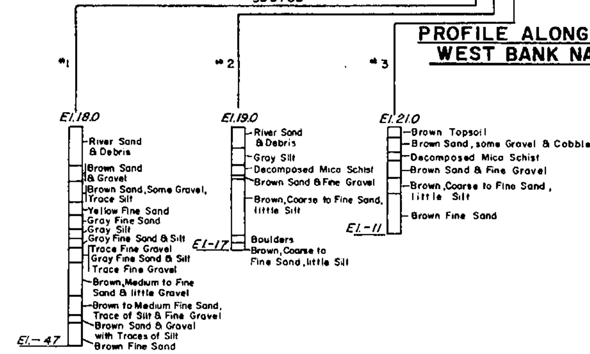
NOTE:
For location of borings #6 and #12, see Plate No. C-4.
Borings made by Conn State Hwy. Dept. 1956.

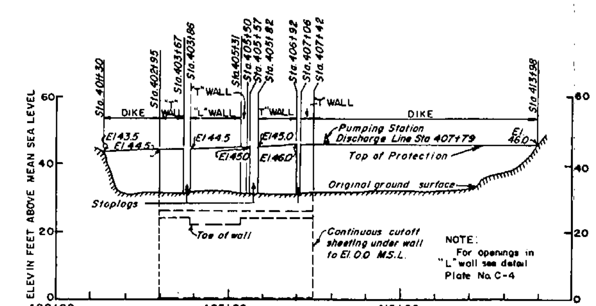




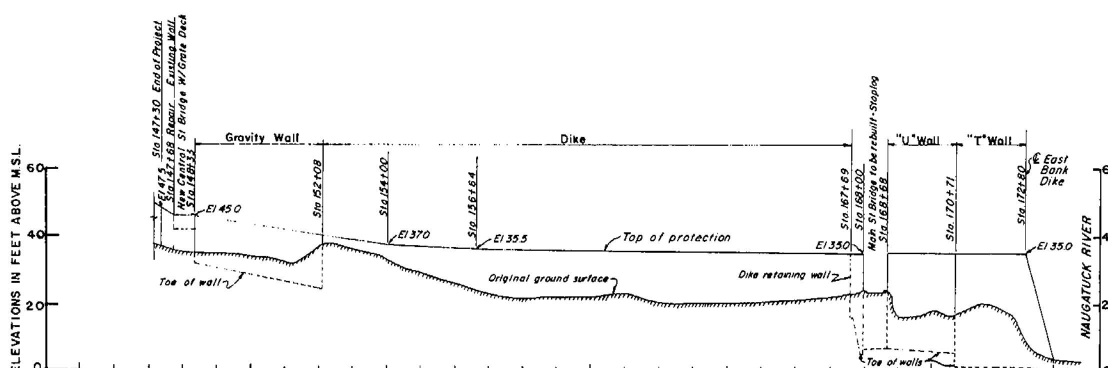
PROFILE ALONG ℄ OF PROTECTION
WEST BANK NAUGATUCK RIVER

NOTE:
For location of borings #1, #2 and #3, see Plate No. C-2.

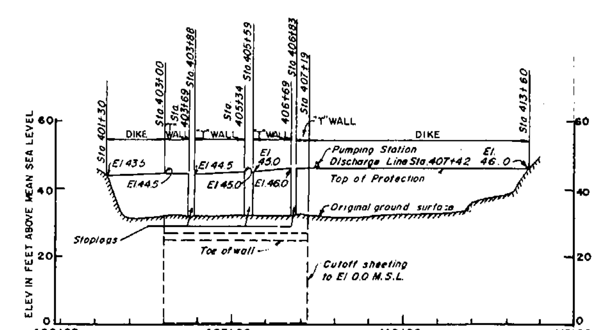




PROFILE ALONG ℄ OF PROTECTION
RIVER ST. AREA - RECOMMENDED PLAN



PROFILE ALONG ℄ OF PROTECTION
NORTH BANK BEAVER BROOK



PROFILE ALONG ℄ OF PROTECTION
RIVER ST. AREA - MOST FEASIBLE PLAN

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY J.A.W. TR. BY A.J.H. CK. BY J.W.D.

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
PROFILES & BORING LOGS

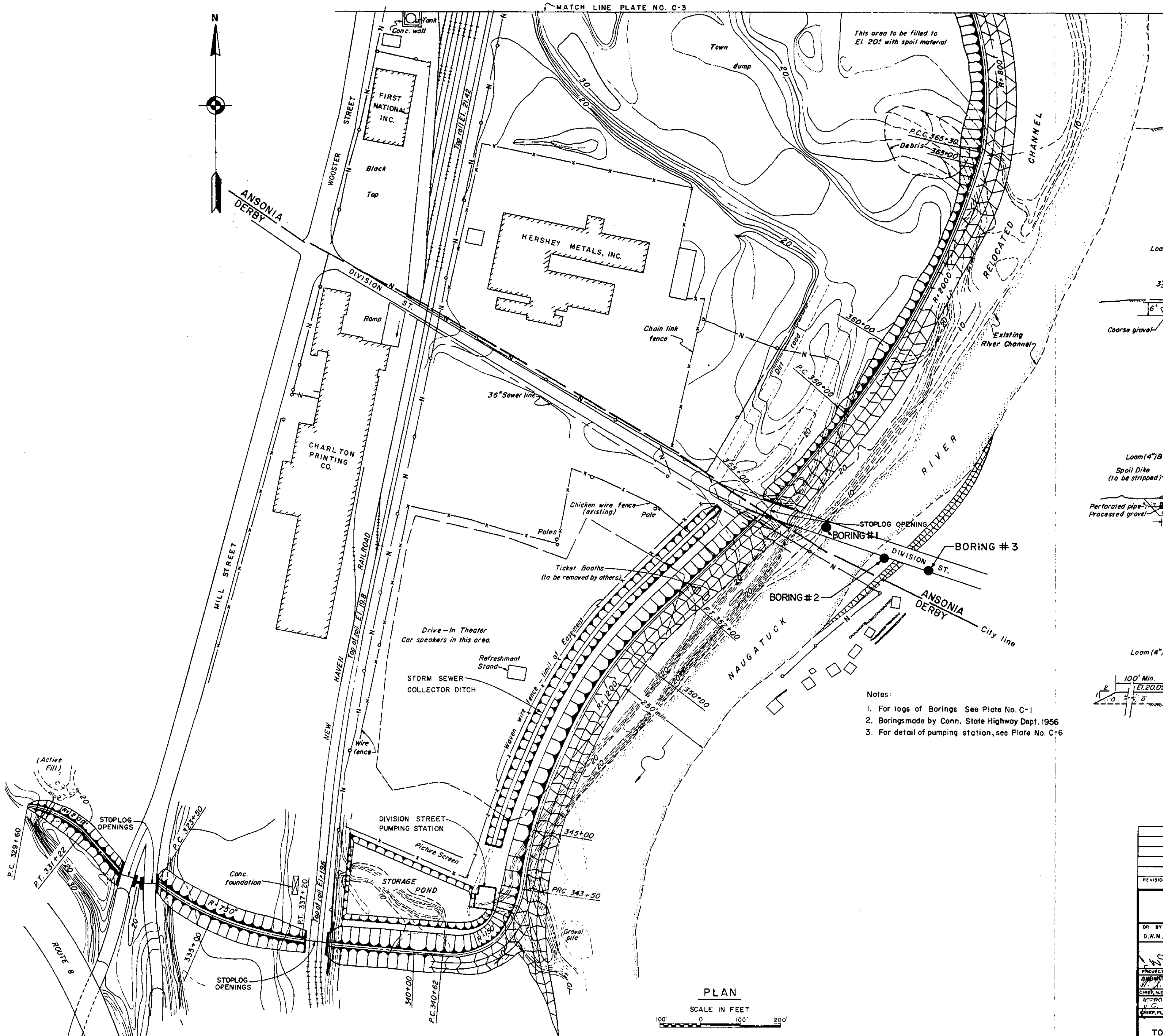
PROJECT ENGINEER
SUBMITTED BY:
CHIEF, N.E. FLOOD STUDIES SECT.
APPROVED:
CHIEF, PLANN. & REP'TS BRANCH
APPROVED:
CHIEF ENGINEERING DIV.

NAUGATUCK RIVER, CONNECTICUT
DATE: APRIL, 1960

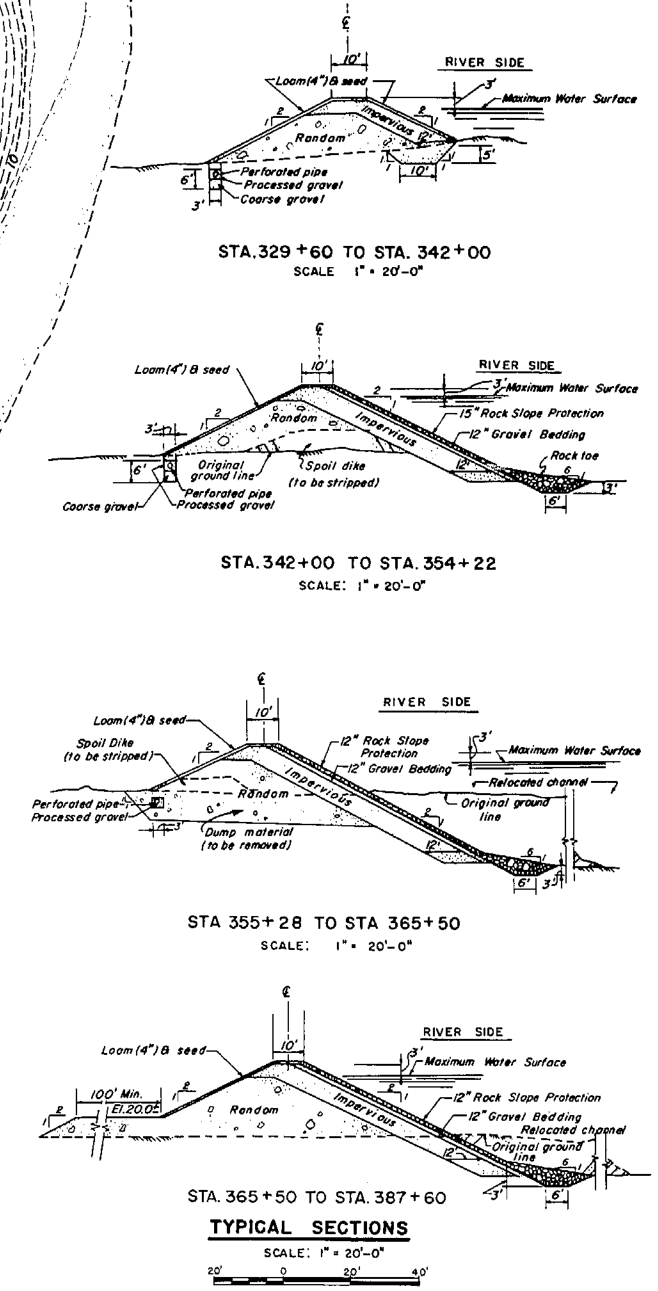
TO ACCOMPANY REPORT
DATED: 13 APRIL, 1960

SCALE: AS NOTED
DRAWING NUMBER
HC-1-1374

PLATE NO. C-1



Notes:
1. For logs of Borings See Plate No. C-1
2. Boringsmade by Conn. State Highway Dept. 1956
3. For detail of pumping station, see Plate No. C-6



REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS

DR. BY	TR. BY	CK. BY
D.W.N.	J. F.	J.W.D.

PROJECT ENGINEER
SUBMITTED BY:
CHIEF, H.E. FLOOD STUDIES SECT.
APPROVED:
CHIEF, PLANN. & RPT'S BRANCH

HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY, CONNECTICUT

LOCAL PROTECTION
DETAIL PLAN I

NAUGATUCK RIVER, CONN.

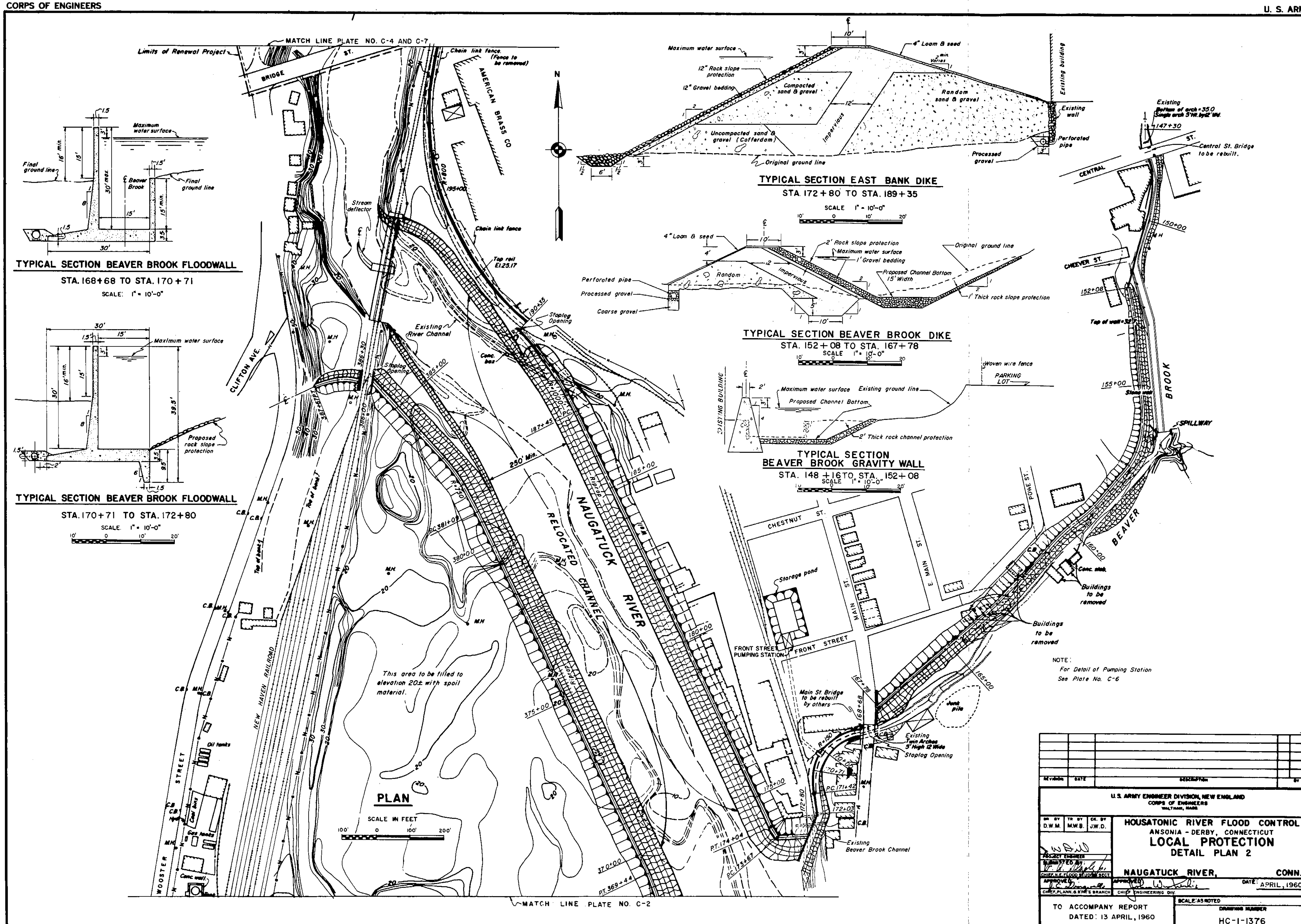
APPROVED:
CHIEF ENGINEERING DIV.

DATE: APRIL, 1960

SCALE AS NOTED

TO ACCOMPANY REPORT
DATED: 13 APRIL, 1960

DRAWING NUMBER
HC-1-1375


MATCH LINE PLATE NO. C-4 AND C-7
Limits of Renewal Project
BRIDGE ST.
AMERICAN BRASS CO
Chain link fence. (Fence to be removed)
N
TYPICAL SECTION BEAVER BROOK FLOODWALL
STA. 168+68 TO STA. 170+71
SCALE: 1" = 10'-0"
TYPICAL SECTION BEAVER BROOK FLOODWALL
STA. 170+71 TO STA. 172+80
SCALE: 1" = 10'-0"
TYPICAL SECTION EAST BANK DIKE
STA 172+80 TO STA. 189+35
SCALE 1" = 10'-0"
TYPICAL SECTION BEAVER BROOK DIKE
STA. 152+08 TO STA. 167+78
SCALE 1" = 10'-0"
TYPICAL SECTION BEAVER BROOK GRAVITY WALL
STA. 148+16 TO STA. 152+08
SCALE 1" = 10'-0"
Maximum water surface
12" Rock slope protection
12" Gravel bedding
Compacted sand & gravel
Random sand & gravel
Impervious
Uncompacted sand & gravel (Cofferdam)
Original ground line
4" Loam & seed
Existing building
Existing wall
Perforated pipe
Processed gravel
Coarse gravel
Proposed Channel Bottom 15' Width
1' Thick rock slope protection
Woven wire fence
PARKING LOT
Proposed Channel Bottom
2' Thick rock channel protection
Final ground line
Beaver Brook
Proposed rock slope protection
CLIFTON AVE.
Stream deflector
Existing River Channel
Stoplog Opening
Chain link fence
Top rail El. 23.17
250' Min.
RELOCATED CHANNEL
NAUGATUCK RIVER
NEW HAVEN RAILROAD
WOOSTER STREET
This area to be filled to elevation 20± with spoil material.
PLAN
SCALE IN FEET
Oil tanks
Gas tanks
Conc. wall
CHESTNUT ST.
MAIN ST.
E. MAIN ST.
FRONT STREET
FRONT STREET PUMPING STATION
Storage pond
Main St Bridge to be rebuilt by others
Existing Twin Arches 5' High 12' Wide
Stoplog Opening
Existing Beaver Brook Channel
CENTRAL ST.
Central St. Bridge to be rebuilt.
CHEEVER ST.
POWE ST.
BEAVER BROOK
SPILLWAY
Buildings to be removed
Junk pile
NOTE: For Detail of Pumping Station See Plate No. C-6
MATCH LINE PLATE NO. C-2
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
D.W.M. M.W.B. J.W.D.
HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
DETAIL PLAN 2
NAUGATUCK RIVER, CONN.
DATE: APRIL, 1960
SCALE: AS NOTED
TO ACCOMPANY REPORT
DATED: 13 APRIL, 1960
HC-1-1376

CORPS OF ENGINEERS

U. S. ARMY

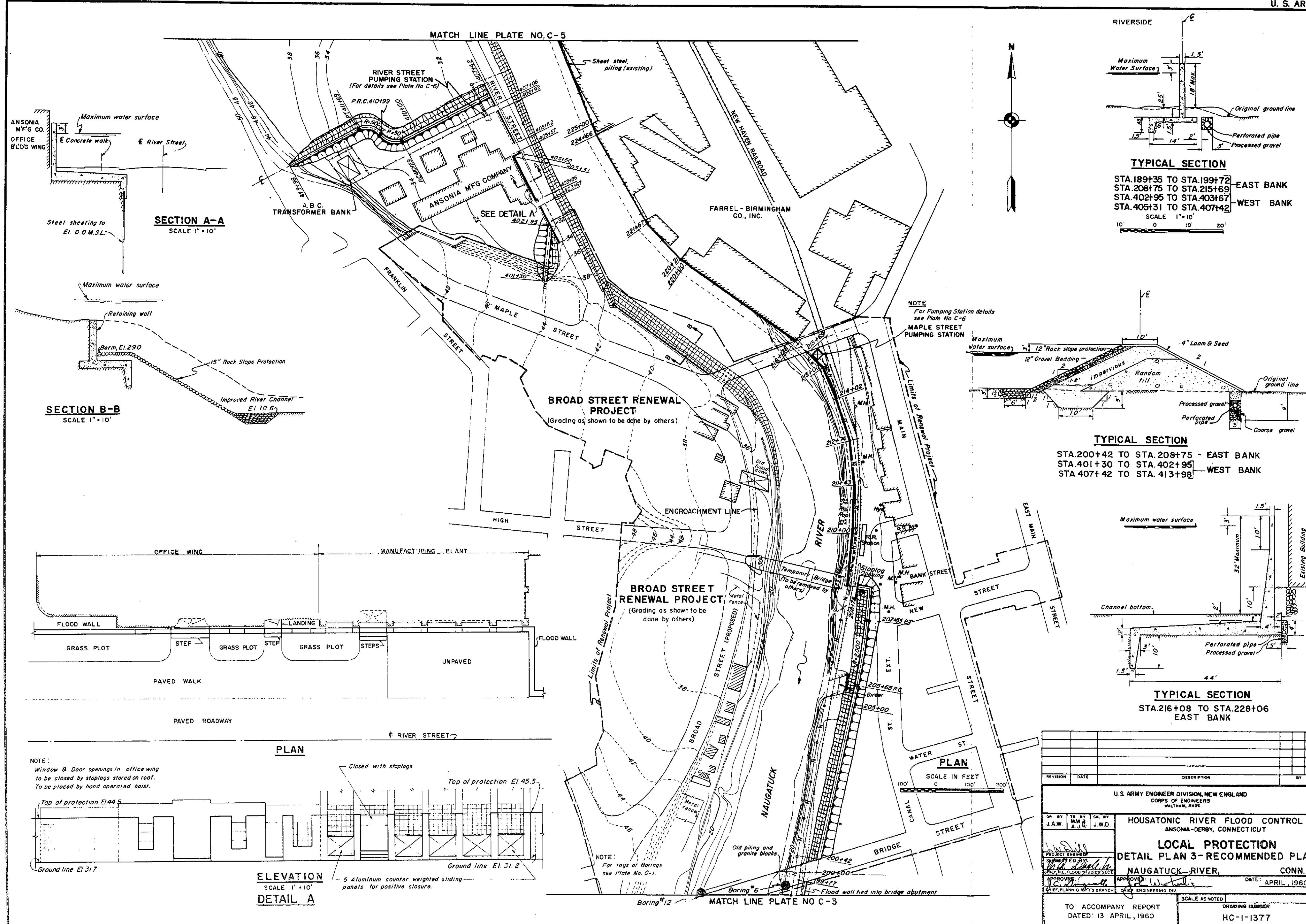
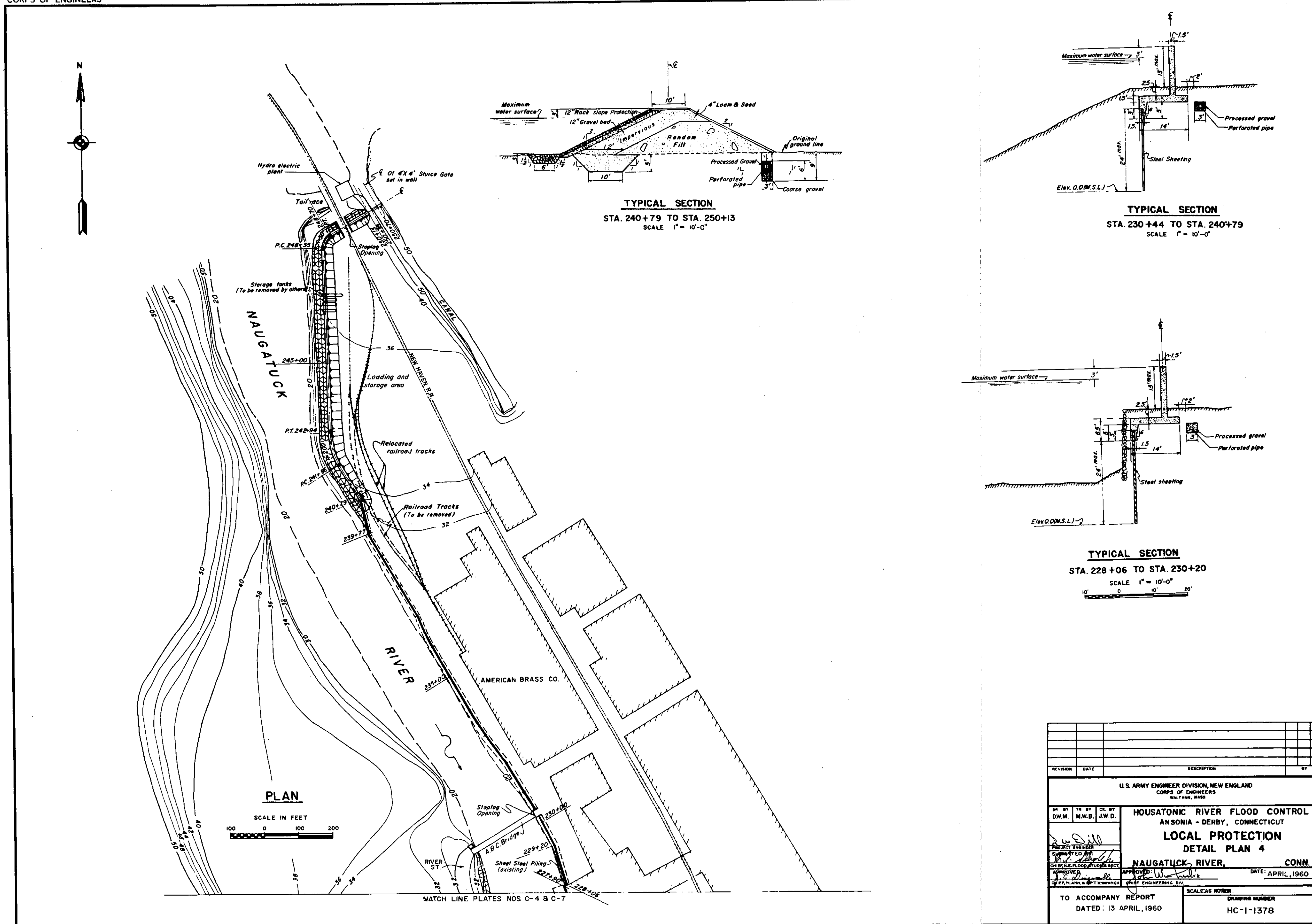


PLATE NO. C-4


N
NAUGATUCK
RIVER
Hydro electric plant
℄ Of 4'X4' Sluice Gate set in wall
Tailrace
Stoplog Opening
P.C. 248+35
Storage tanks (To be removed by others)
CANAL
NEW HAVEN R.R.
245+00
Loading and storage area
P.T. 242+94
Relocated railroad tracks
P.C. 241+96
240+79
Railroad Tracks (To be removed)
239+77
AMERICAN BRASS CO.
234+00
Stoplog Opening
230+00
A.B.C. Bridge
229+20
RIVER ST.
Sheet Steel Piling (existing)
227+90
228+06
MATCH LINE PLATES NOS. C-4 & C-7
PLAN
SCALE IN FEET
100 0 100 200
Maximum water surface
12" Rock slope Protection
12" Gravel bed
Impervious
Random Fill
4" Loam & Seed
Original ground line
Processed Gravel
Perforated pipe
Coarse gravel
TYPICAL SECTION
STA. 240+79 TO STA. 250+13
SCALE 1" = 10'-0"
Maximum water surface
Steel Sheeting
Processed gravel
Perforated pipe
Elev. 0.0(M.S.L.)
TYPICAL SECTION
STA. 230+44 TO STA. 240+79
SCALE 1" = 10'-0"
Maximum water surface
Processed gravel
Perforated pipe
Steel sheeting
Elev. 0.0(M.S.L.)
TYPICAL SECTION
STA. 228+06 TO STA. 230+20
SCALE 1" = 10'-0"
REVISION DATE DESCRIPTION BY
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
DR. BY D.W.M. TR. BY M.W.B. CK. BY J.W.D.
HOUSATONIC RIVER FLOOD CONTROL
ANSONIA - DERBY, CONNECTICUT
LOCAL PROTECTION
DETAIL PLAN 4
NAUGATUCK RIVER, CONN.
PROJECT ENGINEER
SUBMITTED BY
CHIEF, N.E. FLOOD STUDIES SECT.
APPROVED
CHIEF, PLANN. & RPT'S BRANCH
APPROVED
CHIEF, ENGINEERING DIV.
DATE: APRIL, 1960
SCALE: AS NOTED
DRAWING NUMBER
HC-1-1378
TO ACCOMPANY REPORT
DATED: 13 APRIL, 1960

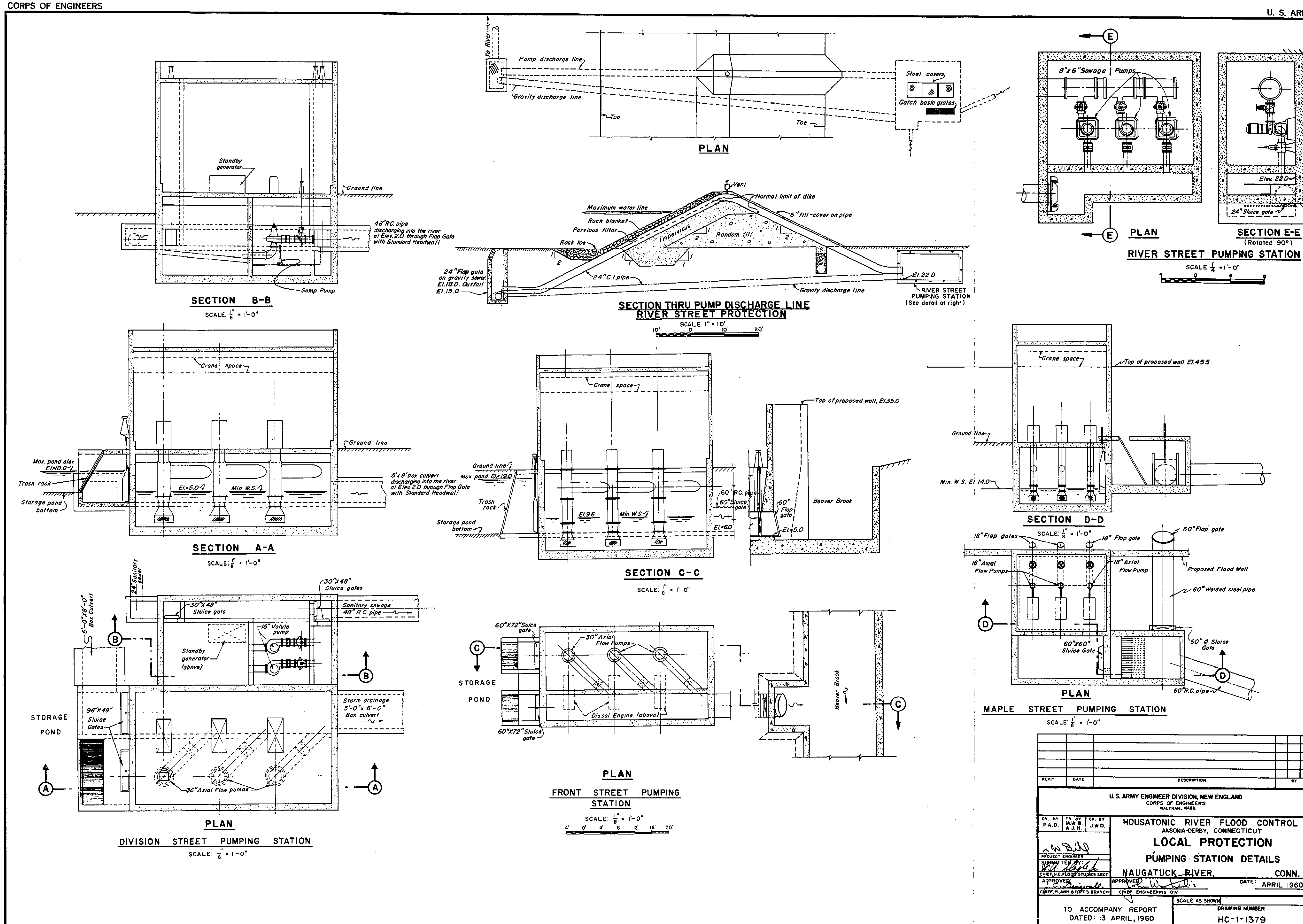

PLAN
To River
Pump discharge line
Gravity discharge line
Toe
Steel covers
Catch basin grates
SECTION THRU PUMP DISCHARGE LINE
RIVER STREET PROTECTION
SCALE 1" = 10'
Vent
Normal limit of dike
6" fill-cover on pipe
Maximum water line
Rock blanket
Pervious filter
Impervious
Random fill
Rock toe
24" Flap gate on gravity sewer El.18.0 Outfall
El.15.0
24" C.I. pipe
Gravity discharge line
El.22.0
RIVER STREET PUMPING STATION (See detail at right)
8"x6" Sewage Pumps
PLAN
SECTION E-E (Rotated 90°)
Elev. 22.0
24" Sluice gate
RIVER STREET PUMPING STATION
SCALE 1/4" = 1'-0"
Standby generator
Ground line
48" R.C. pipe discharging into the river at Elev. 2.0 through Flap Gate with Standard Headwall
Sump Pump
SECTION B-B
SCALE: 1/8" = 1'-0"
Crane space
Ground line
Max. pond elev. El.10.0
Trash rack
Storage pond bottom
El.=5.0
Min. W.S.
5'x8' box culvert discharging into the river at Elev. 2.0 through Flap Gate with Standard Headwall
SECTION A-A
SCALE: 1/8" = 1'-0"
24" Sanitary sewer
5'-0"x8'-0" Box Culvert
30"x48" Sluice gate
30"x48" Sluice gates
Sanitary sewage 48" R.C. pipe
16" Volute pump
Standby generator (above)
STORAGE POND
96"x48" Sluice Gates
Storm drainage 5'-0"x 8'-0" Box culvert
36" Axial Flow pumps
PLAN
DIVISION STREET PUMPING STATION
SCALE: 1/8" = 1'-0"
Crane space
Ground line
Max. pond El.19.0
Trash rack
Storage pond bottom
El.9.6
Min. W.S.
60" R.C. pipe
60" Sluice gate
60" Flap gate
El.6.0
El.5.0
Top of proposed wall, El.35.0
Beaver Brook
SECTION C-C
SCALE: 1/8" = 1'-0"
60"X72" Sluice gate
30" Axial Flow Pumps
STORAGE POND
Diesel Engine (above)
60"X72" Sluice gate
Beaver Brook
PLAN
FRONT STREET PUMPING STATION
SCALE: 1/8" = 1'-0"
Crane space
Top of proposed wall El.45.5
Ground line
Min. W.S. El. 14.0
SECTION D-D
SCALE: 1/8" = 1'-0"
18" Flap gates
18" Flap gate
60" Flap gate
18" Axial Flow Pumps
18" Axial Flow Pump
Proposed Flood Wall
60" Welded steel pipe
60"X60" Sluice Gate
60" ϕ Sluice Gate
60" R.C. pipe
PLAN
MAPLE STREET PUMPING STATION
SCALE: 1/8" = 1'-0"
REV. DATE DESCRIPTION BY
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
DR. BY P.A.D.
TR. BY M.W.B. A.J.H.
CK. BY J.W.D.
HOUSATONIC RIVER FLOOD CONTROL
ANSONIA-DERBY, CONNECTICUT
LOCAL PROTECTION
PUMPING STATION DETAILS
NAUGATUCK RIVER, CONN.
PROJECT ENGINEER
SUBMITTED BY:
CHIEF, N.E. FLOOD STUDIES SECT.
APPROVED:
CHIEF, PLANN. & REP'TS BRANCH
APPROVED:
CHIEF ENGINEERING DIV.
DATE: APRIL 1960
SCALE AS SHOWN
TO ACCOMPANY REPORT DATED: 13 APRIL, 1960
DRAWING NUMBER
HC-1-1379

PLATE NO. C-6

CORPS OF ENGINEERS

U. S. ARMY

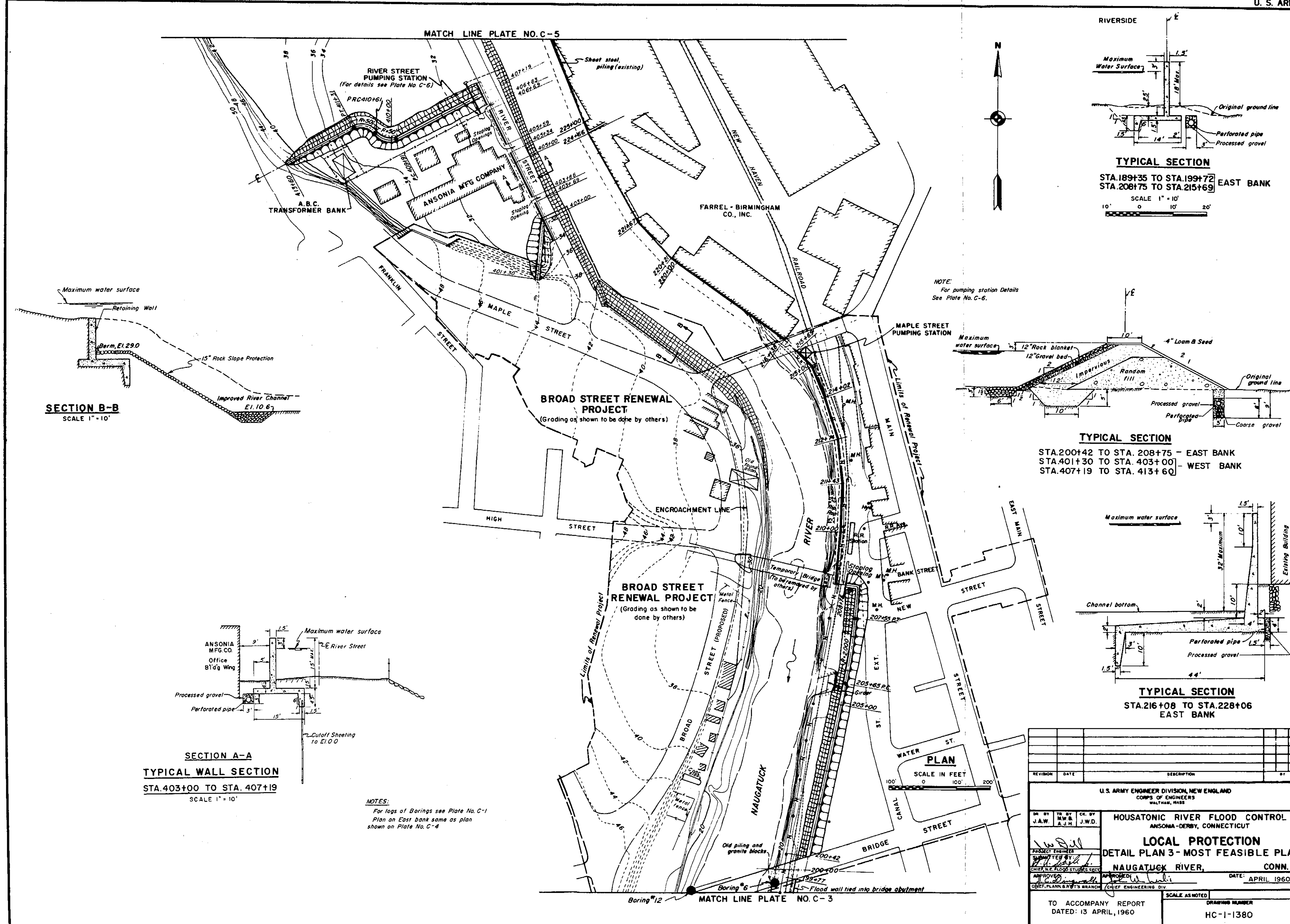


PLATE NO. C-7

APPENDIX D

LETTERS OF COMMENT AND CONCURRENCE

APPENDIX D

LETTERS OF COMMENT AND CONCURRENCE

TABLE OF CONTENTS

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE

59 TEMPLE PLACE
BOSTON, MASSACHUSETTS

ADDRESS ONLY THE
REGIONAL DIRECTOR

NORTHEAST REGION
(REGION 5)
NEW ENGLAND STATES
NEW YORK
PENNSYLVANIA
NEW JERSEY
DELAWARE
WEST VIRGINIA

May 15, 1959

District Engineer
New England Division
U. S. Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear Sir:

From the information submitted by your letter dated April 23, 1959, it does not appear that the local protection works planned for the vicinity of Ansonia, Connecticut on the Naugatuck River would be in any way detrimental to fish or wildlife resources.

We regret the delay in replying to your letter and appreciate the opportunity to review your plans.

Sincerely yours,

M. A. Marston

M. A. Marston, Chief
Division of Technical Services

EXHIBIT D-1

City of Derby

City Clerk's Office

Derby, Connecticut

August 19, 1959

Department of the Army
U.S. Army Engineer Division, New England
Corps of Engineers
424 Trapelo Rd.
Waltham 54, Mass.

Att: Alden K. Sibley
Brigadier General, U.S. Army
Division Engineer

Dear Sir:

At a special meeting of the Board of Aldermen held August 10, 1959, the Board went on record to adopt Plan No. 2, of the Flood Control Project along the Naugatuck River in Derby, as presented by William A. Slagle Jr. and Joseph W. Dill, of the U.S. Army Engineer Corps, and as recommended by City Engineer, George E. Thompson, in preference to an Alternate Plan which was also presented to the Board.

The following members of the Six (6) Man Board, voted for adoption of the plan: President of the Board, Lawrence J. Bartimole, Joseph Stankye, Nebi Hassan, Albert Mizii and Charles E. Ahearn. Board member William Freiheit, voted present.

Very truly yours,

Francis J. O'Connell

Francis J. O'Connell, City Clerk

EXHIBIT D-2

City of Derby

CONNECTICUT

OFFICE OF THE MAYOR

ANTHONY DIRIENZO, MAYOR
MRS. MARGARET VICIDOMINO,
EXECUTIVE SECRETARY

October 14, 1959

Alden K. Sibley
Brigadier General, U.S. Army
Division Engineer
424 Trapelo Road
Waltham 54, Massachusetts

Dear General Sibley:

I am writing to acknowledge receipt of your letter dated May 13, 1959 concerning the plans for flood control protection as effecting the cities of Derby and Ansonia.

With particular reference to the last paragraph on page two of your letter, the Board of Aldermen of the City of Derby at its regular meeting held in September, 1959 authorized me, as Mayor to state that in my opinion the City of Derby will be able and willing to meet the requirements of local cooperation outlined in your letter. As you know the Board of Aldermen has previously approved these plans. It is my feeling that every effort will be made by the necessary boards and agencies of the city to comply with your local cooperation requirements, subject to the necessary appropriation of funds by the Board of Apportionment and Taxation.

Trusting that this is the information that you requested, I am,

Sincerely yours,

CITY OF DERBY

Anthony Dirienzo

Anthony Dirienzo
Mayor

EXHIBIT D-3

City of Ansonia

CONNECTICUT

OFFICE OF THE MAYOR

JOSEPH A. DOYLE, MAYOR
MRS. BERNADETTE McGRATH,
EXECUTIVE SECRETARY

October 27, 1959

Brigadier General Alden K. Sibley
Division Engineer
Corps of Engineers, U. S. Army
New England Division
424 Trapelo Road
Waltham 54, Massachusetts

Re: Local Flood Protection: Ansonia, Conn.

Dear General Sibley:

This letter is in response to your request for assurances of City action, which request accompanied the preliminary study and cost estimates for local flood protection in Ansonia.

Although, as you realize, there remain certain design details to be resolved with the help of your office and the Water Resources Commission before the City accepts in any specific plan, you may be assured that the City of Ansonia concurs in the plans as presented in their general form. As soon as the particular design details have been resolved, the Ansonia Flood and Erosion Control Board will forward its official report and recommendations to me. I shall, of course, add my full and wholehearted endorsement to this report and recommend to the Board of Aldermen that they take speedy and positive action.

I expect, based on the present status of the plans, that the Board of Aldermen will approve the report and plans and authorize a bond issue resolution in December. You will be notified of their action immediately thereafter. Thus official City action on a specific plan can be expected shortly, although, as I said before, you have my assurance now that the City will accept the Plans in the form agreed upon by the Corps of Engineers, The Water Resources Commission and the Ansonia Flood & Erosion Control Board.

Concerning the requirements of local co-operation and local contribution, the City is certainly willing and able to meet its obligations. Our present financial statements indicate an available borrowing authority of over $1,150,000.00; that is, legal debt minus present debt. Moreover, the people of Ansonia, I feel certain, will be willing to contribute their share for flood protection. The general attitude which our people have recently shown indicates a strong and sincere desire for city improvement. Three bond issues, totaling more than $1,600,000.00 were recently approved by the voters. These funds will be applied to schools, redevelopment and the library. Consequently when the first tangible benefits from these projects are seen and appreciated, I am sure that the voters will respond with approval of the local share for flood protection.

Although I know that certain approvals of the local flood protection report must be obtained from the Chief of Army Engineers, the Board of Rivers and Harbors and the Governor of Connecticut, I would hope that the local referendum to approve a bond issue might be scheduled to coincide with our general elections in November, 1960.

I trust this answers the questions you may have concerning the City's position on local flood protection. I hope you will now be able to proceed rapidly with the steps necessary to get this project underway.

If you need any further assurance of this City's desire to proceed with floor control please do not hesitate to get in touch with me. I stand ready, as always to do whatever I can to insure that Ansonia will not again suffer the destruction and tragedy caused by the unruly Naugatuck River in 1955.

Sincerely,

Joseph A. Doyle

JOSEPH A. DOYLE, Mayor
City of Ansonia

EXHIBIT D-4/2

STATE OF CONNECTICUT

WATER RESOURCES COMMISSION

STATE OFFICE BUILDING · HARTFORD 15, CONNECTICUT

December 21, 1959

General Alden K. Sibley, Division Engineer
Corps of Engineers, USA
New England Division
424 Trapelo Road
Waltham, Massachusetts

Dear Sir:

On November 30, 1959, a conference was held in this office with Mr. Leslie and Mr. Restall of your office to further clarify the technical points discussed in your letter of November 25th concerning the Ansonia-Derby local protection project.

In the Derby section of the project the starting elevation is very much dependent upon the evaluation of the head loss which will be caused by debris collecting at the railroad bridge. This could be assumed at 4.0 feet, more or less, but a detailed analysis is not justified because other contributing factors are not available at this time, such as project design flood on the Housatonic River, probability of a specific tide, and evaluation of channel improvements below Division Street. It is assumed that additional information on these factors will be available at the time of final design.

We assume that the comparative benefit-cost ratios shown on page 4 of your letter are based on the continued use of the theatre property for its present purpose and indicates that such usage reduces recurring damages and that the recommended plan substantially reduces the value of the parcel for its present use.

It is our understanding that the project as designed eliminates the value of the proposed improvement at the Maple Street bridge planned by the State and the City and, therefore, such improvement should not be undertaken. We understand that present indications are that relocation of River Street is impractical. It is assumed that if data, available at time of final design, demonstrates that the relocation of River Street is practical, then the necessary changes can be incorporated in the plan.

We assume particularly in view of your letter of December 15 to Ansonia that your final recommendation will be to construct the wall along the land side of River Street at a saving of $380,000.

The statement in your last paragraph, namely, that "any savings in construction costs through detailed studies will be shared proportionally by local interests and the Federal Government" we assume applies to the above comments and we feel that the project as now proposed is satisfactory to the State's interest, and therefore, we approve its submission for authorization.

Very truly yours,

William S. Wise

William S. Wise
Director

WSW:tv

EXHIBIT D-5

ANSONIA FLOOD & EROSION CONTROL BOARD

December 29, 1959

John Wm. Leslie
Chief, Engineering Division
Corps of Engineers
424 Trapelo Road
Waltham 54, Mass.

Dear Mr. Leslie:

The Ansonia Flood & Erosion Control Board met last evening and we were very much pleased with the result of your study of the situation along River Street as it affected the Ansonia Manufacturing Company.

It is our opinion that the wall should be located along the face of the buildings with protected openings at doors and windows.

When the plans and estimates are finalized and accepted by this Board, they will be submitted to the Mayor and Board of Aldermen, with a recommendation that action be taken to make the necessary municipal funds available.

Very truly yours,
ANSONIA FLOOD & EROSION CONTROL BOARD

by C. W. Pearson

C. W. Pearson
Secretary

CWP:O
cc: Mayor Joseph Doyle
Mr. Richard Mace

EXHIBIT D-6

ANSONIA FLOOD & EROSION CONTROL BOARD

Ansonia, Connecticut

February 29, 1960

John W. Leslie
Chief, Engineering Division
Corps of Engineers
424 Trapelo Road
Waltham 54, Mass.

Dear Mr. Leslie:

At a meeting of the Ansonia Flood & Erosion Control Board held on February 26, it was the unamimous opinion of the Board that the plan of protection for the Ansonia Manufacturing Company as shown on the plan sent to us on January 18 be approved and adopted.

It is understood that the additional cost will be included in the contribution by the City for the entire Project.

Very truly yours,
ANSONIA FLOOD & EROSION CONTROL BOARD

by
C. W. Pearson, Secretary

cc: Mr. Richard Mace
Mayor Joseph A. Doyle

EXHIBIT D-7

ATTACHMENT II

ANSONIA-DERBY LOCAL PROTECTION

NAUGATUCK RIVER
CONNECTICUT

Information Called for By Senate
Resolution 148, 85th Congress
Adopted 28 January 1958

INFORMATION CALLED FOR BY SENATE RESOLUTION 148 85TH CONGRESS, ADOPTED 28 JANUARY 1958

1. PROJECT DESCRIPTION AND ECONOMIC LIFE

a. Recommended project. The recommended project for flood protection for the city of Ansonia and a small contiguous area of Derby, Connecticut consists of three elements:

(1) Protection along the west bank of the Naugatuck River in Ansonia and Derby, in the vicinity of Division Street (the town line), consisting of earth dikes, stoplog structures, a pumping station for storm water and sanitary sewage, and appurtenant structures;

(2) Protection of a small area in Ansonia along the west bank of the Naugatuck River upstream from Maple Street, consisting of earth dikes, concrete flood walls, stoplog structures and a small pumping station to handle interior storm water; and

(3) Protection along the east bank of the Naugatuck River, lying wholly within Ansonia, consisting of earth dikes, concrete flood walls, stoplog structures, two pumping stations to handle interior storm water runoff, and appurtenant structures. The east bank protection extends up Beaver Brook, a tributary of the Naugatuck River in the lower section of Ansonia to Central Street and consists of earth dikes and concrete flood walls along the right bank of Beaver Brook and reconstruction of the Main Street and Central Street bridges over Beaver Brook.

Complete descriptions of this project are given in Section XV of the main report and in Appendix C and shown on plates appended to Appendix C.

b. Alternative projects. Several alternative layouts were studied in detail. One plan, referred to as the most feasible plan, was less expensive than the recommended plan while realizing essentially the same benefits. However, local interests prefer the recommended plan and have expressed willingness to pay the additional cost thereof. Descriptions of these alternatives are given in Section XV of the main report and in Appendix C.

c. Economic life. The estimated project life used in the economic analyses in the report is fifty years.

2. PROJECT COSTS

Project first costs are based on average bid prices for similar work in the same general area adjusted to 1960 price levels. Annual charges in the report are based on interest on the investment and amortization over the 50-year assumed project life to which are added amounts for maintenance and operation of the project, interim replacement of equipment having an estimated life less than 50 years, and loss of productivity on land taken for project purposes. Interest rates are 2.5 percent for Federal costs, 3.0 percent for non-Federal costs and 6.0 percent for loss of productivity. First costs and annual charges are detailed in Appendix C. Table II-1, appended to this supplement, shows a comparison of first costs and annual charges for the recommended and the most feasible projects based on 50-year and 100-year economic life.

3. PROJECT BENEFITS

Estimates of recurring losses in the Ansonia area are predicated on a continuation of use at the present level. Growth within the flood area which might increase the recurring loss during the economic life of the project is not anticipated, except as covered by enhancement benefits. Therefore, the average annual flood damage prevention benefits would be the same for an economic life of either 50 or 100 years. Enhancement benefits are based on an average annual return on increased land values or rentals of vacant space and likewise do not change if applied against either a 50- or 100-year economic life.

Intangible benefits would accrue since the project would provide an atmosphere of greater stability and confidence in the productive heart of the city. Threats of disease, emergency evacuation measures, and much of the threat to life posed by severe flooding would be virtually eliminated. In addition, protection will be provided for two major metal industries, both of which were important contributors to the national defense.

4. BENEFIT-COST RATIOS

Benefit-cost ratios for the recommended and most feasible plans are shown on Table II-1. It will be noted that use of a 100-year economic life results in raising the benefit-cost ratio for the alternative projects proportionately.

5. PHYSICAL FEASIBILITY AND COST OF PROVIDING FOR FUTURE NEEDS

The recommended plan would provide protection for now vacant and flood-prone areas and buildings which will undoubtedly be utilized if protection is provided. In this respect the plan takes cognizance of the future needs of the area in making available sorely needed industrial and commercial space. The cost-sharing feature for providing for future needs is recognized through enhancement benefits and the appropriate cash contribution required from local interests.

On the east bank in the area which would be protected, the City of Ansonia is planning an Urban Renewal project. It is understood, through the Director of the Ansonia Redevelopment Agency, that formal application has been made to the Federal Housing and Home Finance Administration but that approval has been withheld pending submission by the Corps of Engineers of a recommendation for protection of the area against flooding. As noted in the main report, no tangible benefits have been ascribed to protection of the proposed redevelopment since no definitive plans for the area have yet been approved.

6. ALLOCATION OF COSTS

Both the recommended projects and the alternative projects studied are for flood control only and therefore no allocations of costs among project purposes are required. Apportionment of costs between Federal and non-Federal interests was based on standard prescribed procedures wherein non-Federal interests bear 50 percent of the amount derived by applying to the total cost of the project the ratio of enhancement benefits to total project benefits. Derivation of these allocations is given in Appendix C and would be unchanged by variation in estimated project life.

7. EXTENT OF INTEREST IN PROJECT

Officials of the City of Ansonia have evidenced intense interest in flood protection for the city and have indicated that the City would be willing and able to provide the necessary assurances of local cooperation, including the cash contribution, when required. The City of Derby has also indicated concurrence in the project and willingness to comply with the requirements of local cooperation. The Naugatuck Valley River Control Commission and the Connecticut Water Resources Commission have also concurred in the recommended projects. Firm assurances of local participation will be obtained after authorization but prior to initiation of construction. Estimated costs to Federal and local interests are given in paragraph 47 of the report.

8. REPAYMENT SCHEDULES

There are no reimbursable functions incorporated in any of the studied projects. The non-Federal cash contribution required will be obtained prior to initiation of construction.

9. EFFECT OF PROJECT ON STATE AND LOCAL GOVERNMENTS

The studied projects will have little adverse effect on present State and local governmental services since the areas to be protected are already largely served by existing utilities, sewers, police and fire protection, schools and other public services. Anticipated construction in the enhancement areas for industrial and commercial purposes will not appreciably increase the need for such services. Construction of the proposed Urban Renewal project, which is dependent upon construction of the recommended protection project, may necessitate increasing some of the public services such as schools. However, it is not considered that this increase should be charged against the project since urban renewal is apparently justified on its own merits and it is merely coincidental that the location selected is within the area to be protected.

Tax revenues should be increased due to construction of the recommended project, based on increased values of properties no longer subject to flooding and new construction in the now flood-prone areas afforded protection. The loss of taxes on land required for project purposes is considered to be negligible and will undoubtedly be more than offset by higher valuations on property afforded protection. For economic analysis, the net change in tax revenue has been assumed to be zero, which is considered to be conservative.

10. PROPOSED INCREASES IN APPROPRIATIONS

The recommended project would increase the necessary appropriations for continuation of the construction of flood control projects in the Housatonic River Basin by $4,850,000. With approval of the project recommended in this report, the basinwide flood control plan for the Housatonic River, together with the status of each element, is given in the following table:

Project	Present Status
Dams and Reservoirs	
Thomaston	Under construction - completion 1961
Hall Meadow	Authorized - under design
East Branch	Authorized
Northfield Brook	Recommended in report dated 30 June 1958
Hancock Brook	Recommended in report dated 30 June 1958
Black Rock	Recommended in report dated 30 June 1958
Hop Brook	Recommended in report dated 30 June 1958
Local Protection	
East Branch Naugatuck River, Torrington (P. L. 685)	Completed
West Branch Naugatuck River, Torrington (P. L. 685)	Under construction
Naugatuck River Waterbury-Watertown (P. L. 685)	Under design
Ansonia-Derby	Recommended in this report

Design heights for the Ansonia-Derby local protection project are predicated on the standard project flood modified by authorized and recommended reservoirs with three feet of freeboard. After completion of the Thomaston Reservoir but before completion of the recommended reservoirs, the Ansonia-Derby project would provide protection against the maximum flood of record (70 percent of the standard project flood) without encroaching on the freeboard and against 80 percent of the standard project flood with no freeboard. In order, therefore, to realize full benefits from the project recommended in this report, the dams should be constructed first or concurrently with the local protection projects. However, in view of the high degree of protection provided by the local protection project even before construction of the recommended reservoirs, as discussed above, initiation of these projects should not be made contingent upon initiation or completion of the reservoir projects.

TABLE II - 1

COST ALLOCATION - 50 yr. and 100 yr. Life —— RECOMMENDED AND MOST FEASIBLE PLANS

Item	Recommended Plan 50 yr. life	Recommended Plan 100 yr. life	Most Feasible Plan 50 yr. life	Most Feasible Plan 100 yr. life
First Cost				
Federal	$4,870,000	$4,870,000	$4,870,000	$4,870,000
Non-Federal				
Lands etc.	300,000	300,000	300,000	300,000
Cash Contribution	850,000	850,000	770,000	770,000
Total	1,150,000	1,150,000	1,070,000	1,070,000
Total	6,020,000	6,020,000	5,940,000	5,940,000
Annual Charges				
Federal	176,000	136,300	176,000	136,300
Non-Federal				
Financial	56,800	50,300	53,500	47,500
Economic	59,100	52,600	55,800	49,800
Total				
Financial	232,800	186,600	229,500	183,800
Economic	235,100	188,900	231,800	186,100
Annual Benefits				
Flood Prevention	206,000	206,000	206,000	206,000
Enhancement	84,000	84,000	84,000	84,000
Total	290,000	290,000	290,000	290,000
Benefit-Cost Ratio	1.2	1.5	1.3	1.6